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DIRECT MEASUREMENT OF BOND STRESSES
IN REINFORCED CONCRETE

H. L. Kasten

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T H E S I S

Direct Measurement of Bond Stresses
in Reinforced Concrete.

Submitted in partial fulfillment of the requirements
for the Degree of Master of Science,

under the direction of
R.N. McMenus, M.Sc.

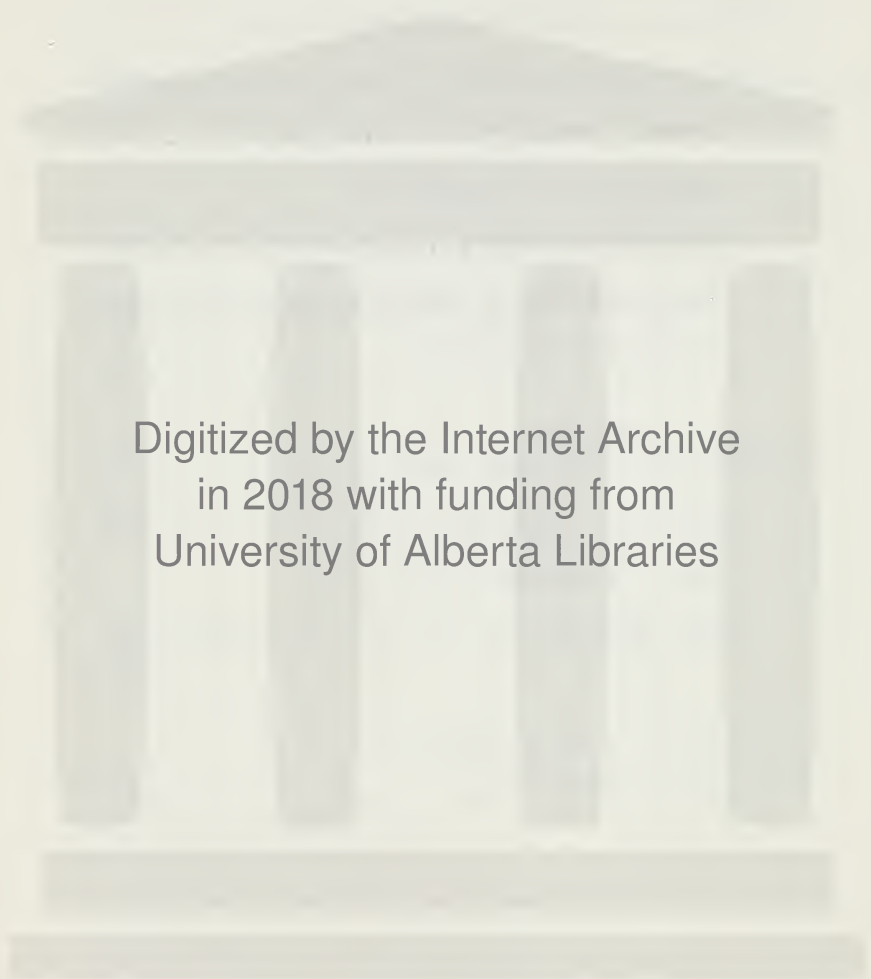
University of Alberta,
April, 1949.

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INTRODUCTION

The action of the bond between concrete and reinforcing steel, on which the effectiveness of reinforced concrete so largely depends, is not satisfactorily understood for careful design purposes. In this work an attempt has been made to learn more of the true action of bond, basing the study on the relatively new method of strain measurement using electric resistance wire strain gages. Furthermore, some knowledge has been gained on the technique of using electric strain gages inside concrete, especially in laboratory tests.

An immense amount of work has been done in the past concerning the bond between concrete and steel. Previous investigators have studied, mostly by laboratory tests, the effects of numerous variables such as strength of concrete, surface texture of bar, vibration, etc., which might influence the effectiveness of bond development. A great deal has been learned about the various factors studied, and considerable controversy has arisen. However, little is said in the literature concerning the true nature of bond.

The present state of knowledge indicates that much is known quite conclusively concerning the effect of physical variations in both steel and concrete on the total bond developed. Little is known regarding the distribution of bond stresses along a stressed steel bar in concrete, for the reason that previous investigators could not "get inside" the concrete to "see" what was going on.

An excellent summary of the past knowledge on bond is given in an article (1)^{*} to the A.C.I. by Messrs. Gilkey, Chamberlin, and Beal. These investigators list the factors which may influence the potential bond resistance in concrete, and then state 20 current leading questions regarding bond, and some answers thereto. A few of the questions are answered rather conclusively, and this may

* Refers to bibliography.

be taken as knowledge consolidated up to the present.

The established principles are briefly as follows:

(1) Influence of Strength of Concrete: "For current concrete mixes, bond resistance does not increase in proportion to strength of concrete.

(2) Effect of Length of Embedment on Anchorage Bond:

Up to $\frac{l}{d} = 24$ the total bond resistance increases with added length of embedment, but beyond 24 diameters extra embedment develops little added resistance.

(3) Comparison of deformed versus plain bars

Initial slip is at only slightly higher load for deformed bars than for plain bars. However, the factor of safety against sudden failure is considerably higher for deformed bars.

(4) Effect of vibration of concrete around the bars

It has been fairly definitely shown that vibration up to nine hours after pouring is beneficial to the total bond which is developed by the bars. The likely reason is that vibration reduces the water gain which tends to form around bars shortly after pouring.

Of the many questions to which the answers are

incomplete, the following few are most important:

- (1) What is the effect of compression versus tension in the surrounding concrete?
- (2) How effective are hooks in developing anchorage bond?
- (3) What is the true distribution of anchorage bond stresses?
- (4) How do factors of safety in bond given by current specifications compare with factors of safety against failure of the steel and concrete in direct stress?
- (5) What is a suitable type of bond test?

The authors and others favor pull-out type tests over beam tests. They state that the needed bond test specimen should be either a pull-out or push-out specimen with suitable provision for the elimination or evaluation of the length or $\frac{L}{D}$ variable. They emphasize that the results may be relative rather than absolute.

It may be noticed that many of the unanswered questions concern internal conditions of the reinforced concrete. One method of getting at the internal stresses has previously been to measure concrete strains on the outside of the specimen. The results have been inconclusive, probably because the measurements were taken at a distance removed from where the action occurred. In the past few years, with the introduction of the electric strain gage, it has

become possible to measure strains at many previously inaccessible places. Methods have been developed for measuring strains with these gages even inside concrete, and it was decided to apply one of these methods in this work to the problem of studying the action of bond stresses inside reinforced concrete.

III.

OBJECTIVES AND METHODS

With the questions of the previous section in mind a list of objectives to be studied in this work was decided upon. The procedure as listed below was to be followed in studying bond stresses:

- (1) To measure the distribution of steel stress in pull-out tests of plain bars embedded in concrete cylinders. As the variation in steel stress can only be caused by the bond between the concrete and steel, a knowledge of anchorage bond distribution should be gained from such tests.
- (2) To compare the anchorage of bars in compression concrete with the anchorage of bars in tension concrete. Both pull-out tests and a reinforced concrete beam test would be used. By varying the support conditions of the concrete surrounding the bar in pull-out tests, anchorage in either compression or tension concrete can be obtained. Also, anchorage under either type of concrete stress can be obtained in a continuous two-span reinforced concrete beam.
- (3) To study the bond stresses due to the shearing forces in the two-span reinforced concrete beam mentioned above. This would be done by measuring

the intensity of stress in the reinforcing bars at various points along the bars.

Before the actual measurement of steel stresses in test specimens could be carried out, it was necessary to learn the technique of waterproofing electric strain gages which are to be embedded in concrete. As several methods had previously been developed by others, it was decided to choose the one method most suited to the projected work and test it thoroughly, making improvements if possible. The question of the applicability of the method to field tests of structures was to be kept foremost in mind. A description of the technique finally used and a discussion of its values and weaknesses is given in Appendix A of this report.

IV.

THEORY

The following is a brief review of the theory of bond stresses, as well as a summary of design practice according to the Canadian Engineering Standards Association specifications.

(a) Nature of bond

The true nature of the bonding action of concrete to steel is not known. It has, however, been well established that bond strength can be separated into two distinct elements - adhesive resistance and sliding resistance. The true action of the adhesive resistance is not understood, except that it must be over-come before the sliding resistance comes into effect. Once sliding, or slippage, of the steel through the concrete begins, it is thought that the bond resistance is due to the irregularities in the surface of the bar moving past the corresponding conformations in the concrete. It has been shown for ordinary concrete that the ultimate value of sliding resistance is somewhat higher than the maximum adhesive resistance. However, in practice, the adhesive resistance is the more important of the two, since, once it is over-come, the deformations of structures allowed by the slipping steel are excessive.

(b) Bond Test Practice

Present day practice, based on simple pull-out and

beam tests, assumes that the maximum adhesive strength of concrete on bars is roughly proportional to the compressive strength of the concrete. This result is more relative than absolute since it depends on simplified assumptions regarding the distribution of the bond stresses. For instance, in pull-out tests the bond is assumed to be uniformly distributed on the bar surface over the full length of embedment. Also in both pull-out tests and beam tests an arbitrary value of total slip of the bar (about 0.0001 inch usually) is taken to indicate the over-coming of adhesive resistance. Thus allowable bond stress values based on the above premise are only roughly correct. Furthermore it has lately been shown (see History) that for higher strength concretes bond resistance does not increase in proportion to concrete strength, so that a new criterion for bond strength is necessary.

(c) Bond Design Practice

Two types of bond stress computation are necessary in designing for bond. These involve checking

- (a) Anchorage bond, and
- (b) Bond due to shearing forces.

Anchorage bond stress results from embedding the bar in concrete for some distance beyond a point where the bar is needed, so that the bar may maintain a certain stress at that point. Assuming a uniformly distributed anchorage bond stress, the length of embedment required to develop a

CHAPTER I. THE DISCOVERY OF AMERICA.

THE first discovery of America was made by Christopher Columbus in 1492.

He sailed from Spain on the 3rd of September, and after a long voyage, he reached the island of San Salvador on the 12th of October.

He then sailed on to the mainland, and discovered the great bay of San Pedro de Mucilipon.

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stress of f_s in a bar of diameter d is given by

$$l = \frac{f_s d}{4u}, \text{ where } u \text{ is the working anchorage stress.}$$

The U.S.A. Code allows an anchorage bond stress equal to $200 - \frac{240,000}{f_c}$ p.s.i., where f_c is the 28 day compressive strength of the concrete. This value is for plain bars having no hooks, with the $\frac{l}{d}$ ratio not to exceed 25. Another stipulation is that the bar is to be anchored in concrete not subjected to appreciable tensile stress. If a hook is used as anchorage, the allowable bond stress on its surface is 125% of the value given above. No special distinction is made for bars in compression, except that anchorage shall consist of straight embedment.

Bond stresses due to shearing forces in beams result wherever there is a variation in stress along a bar. The difference in stress between any two points in the bar is transferred from the bar to the concrete by bond. Since the variation in bar stress is proportional to the rate of change of bending moment at any section, which in turn is equal to the shearing force at that section, then the bond stresses on stressed bars in beams are proportional to the shearing forces. The value of these bond stresses at any point may be found from $u = \frac{V}{jd \Sigma o}$ for members of constant depth.

The U.S.A. code allows a working bond stress of this type equal to $400 - \frac{480,000}{f_c}$ p.s.i. The value is higher than that for anchorage bond because the maximum value will usually exist only over short portions of the bar, and possibly only

for short periods of time.

The above considerations were kept in mind in studying the results of the pull-out tests and beam test that follow.

THREE PULL-OUT TESTS

V.

(a) Purpose.

As pull-out tests of bars are commonly used in checking the bond strengths developed in reinforced concrete, it was decided to apply the technique of SR-4 strain gage measurement to several pull-out specimens. The following objectives were to be investigated:

- (1) The distribution of bond stress along the embedded length of the bar.
- (2) The effect produced by the kind of concrete stress surrounding the bar on the distribution of bond stresses, i.e. compression concrete as compared to tension concrete.
- (3) The effect of varying the $\frac{L}{D}$ ratio of the bar on the distribution of bond stress.

(b) Method.

Three pull-out tests using electric strain gages were carried out in order to make a reconnaissance survey of the above objectives. All three tests were on a 5/8" intermediate grade bar embedded in a 6" diameter concrete cylinder. The design strength of the concrete was 2500 p.s.i. throughout. Two of the tests were with the concrete in compression while one was with the concrete in tension. The details of the three tests may be seen in Figure 1, page 13. Special mention of the reasons for the strain gage locations

is required. With gages placed at various points along the rod, the total stress at successive sections can be measured. Differences in stresses between any two sections are due to the bond developed between those two sections. Then the difference in stress divided by the available bond area between the sections gives the average intensity of bond stress between the two sections. By plotting a sufficient number of these average intensities of bond stresses against their locations along the bar, a curve showing the distribution of bond stress will be obtained.

It will be noticed that with the gages located along the bar as shown in Fig. 2, the bonded area consists of a series of short segments separated by short lengths of bar over which no bond can be developed. An important question is whether the bond developed by a series of separated segments of bar will be the same as for a continuous embedment of equal length or not; and also, what effect will the size of the waterproofing housing have on the distribution of bond stress, since stress concentrations in the concrete are likely.

From the result of Tests 2 (a) and 2 (b) it would appear that the presence of the gages did not adversely affect the total adhesive resistance of the concrete. In these tests the total pull at first slip was about 5300 pounds for the bar with gages, and about 4500 pounds for an equal continuous length of bar embedded without gages. However, in test 2 (a) the presence of gages evidently

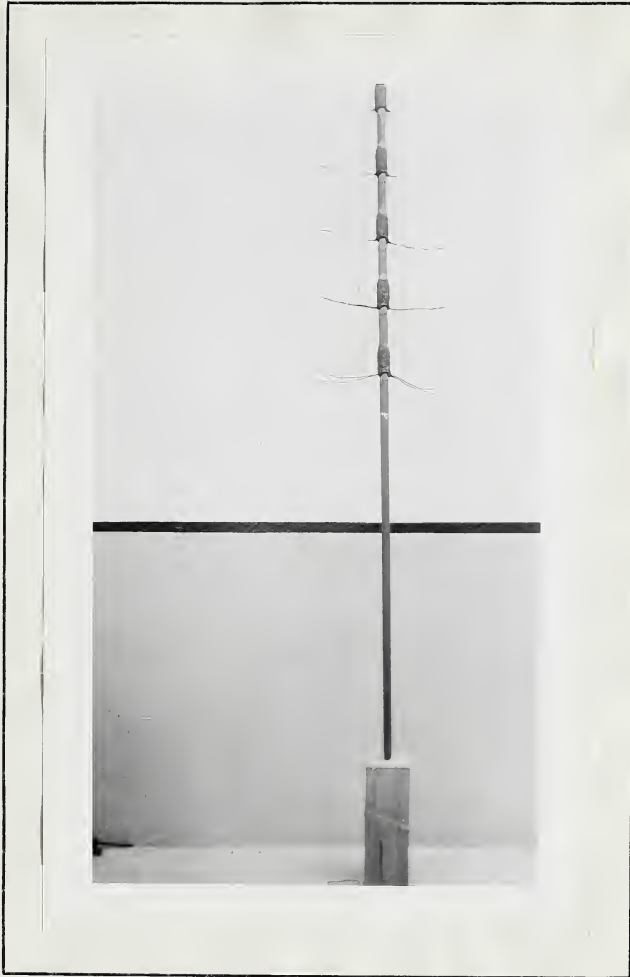


FIGURE 2

WATER PROOFED GAGES ON BAR

FOR PULL-OUT TEST NO. 3



FIGURE 2a.

PULL-OUT TEST No. 3

produced a concentration of bond stress near the unloaded end of the bar at high loads. This was probably due to wedging action of the rubber housings. There is also some evidence of this at high loads in tests 1 and 2.

(c) Tests

The tests were carried out after the concrete had cured for 28 days under water, except in the case of tests 2 (a) and 2 (b) where the cylinders were cured for 20 days only. Auxiliary strength test cylinders were tested on the same day.

Before the actual test the strain gages were all checked in the usual manner. Thereafter loads were applied to the bars very slowly in about 500 pound increments. The gages were read immediately after each test load was reached this load being held constant during the reading period. In tests 1 and 2 an Ames dial at the unloaded end of the bar was read to indicate end-slip.

The original data sheets taken for the three pull-out tests may be found in Appendix B of this report.

(d) Results.

Two types of graph were plotted from the computed data for each of the three tests. The first type for each test shows the variation of unit stress in the bar along the embedded length of the bar, plotted for various applied loads. (See graphs that follow). In plotting these graphs

the portions of the bar covered by the rubber housing were assumed to be of zero length. The second type of graph shows the distribution of anchorage bond stress along the embedded length. Here again the gage locations were considered as points, and the average bond intensity over each segment of the bar was plotted as the value at the mid-point of the segment.

It may be noted here that the value of the Modulus of Elasticity for steel was taken as 30×10^6 p.s.i. in all computations. An actual test on a $5/8 \phi$ bar using four SR-4 strain gages gave value for E of 30.0×10^6 p.s.i.

(e) Interpretation.

Before discussing the results of the pull-out tests it must be kept in mind that, with but three tests performed one can gain only an indication of what happens inside the concrete.

The various graphs and table I on the pull-out tests give some new information on the following features of bond:

(1) Distribution of anchorage bond stress.

For low loads, where the values obtained are the most reliable, the intensity of bond stress is very high at the loaded end of the bar, and then falls off rapidly to a negligible value at some distance from the unloaded end. This holds true in general for the concrete in either compression

or tension.

In no case is there a uniform distribution of bond stress along the bar, although in Test No. 1 the distribution becomes nearly uniform at a high slippage load.

(2) Effect of the stress in the concrete on the bond distribution

Disregarding Test No 2, where the influence of the rubber housings is suspected, it may be seen that a much higher intensity of bond is developed for any given load when the concrete is in tension as compared to when it is in compression. The distribution of bond stress does not, however, "even-out" nearly as much at high loads with the concrete in tension as it does with the concrete in compression.

The ultimate load on the bar in Tests 1 and 3, with a comparable length of embedment, was nearly the same, being about 13,000 pounds for the concrete in compression, and 12,750 pounds for the concrete in tension. At these loads the bar was being pulled slowly through the concrete, and the effect of the rubber housings would be great.

(3) Effect of varying the $\frac{l}{d}$ ratio.

Unfortunately little can be concluded from Tests 1 and 2, since Test No. 2 is of doubtful

value. However, one can see that at very low loads there is good agreement in the shape of the bond distribution curves for each test, as well as in the actual values of bond developed. Thus for the 1500 pound load, the intensity of bond at the loaded end was 190 p.s.i. for Test No 1 and 204 p.s.i. for Test No 2.



(f) Discussion

The development of a higher intensity of bond stress with the concrete in tension than with the concrete in compression is interesting. An explanation possibly lies in the lateral deformations of the concrete due to Poisson's effect. With the concrete in tension the concrete would contract laterally, and thus "close" on the steel, whereas with the concrete in compression it would tend to move away from the rod. Since the rod itself is in tension in both cases, the effect of its lateral contraction would be the same throughout. It may be pointed out, however, that with the cylinder reinforced with $4 - \frac{5}{8} \phi$ bars as in Test No. 3, the tension stress in the concrete would be rather small.

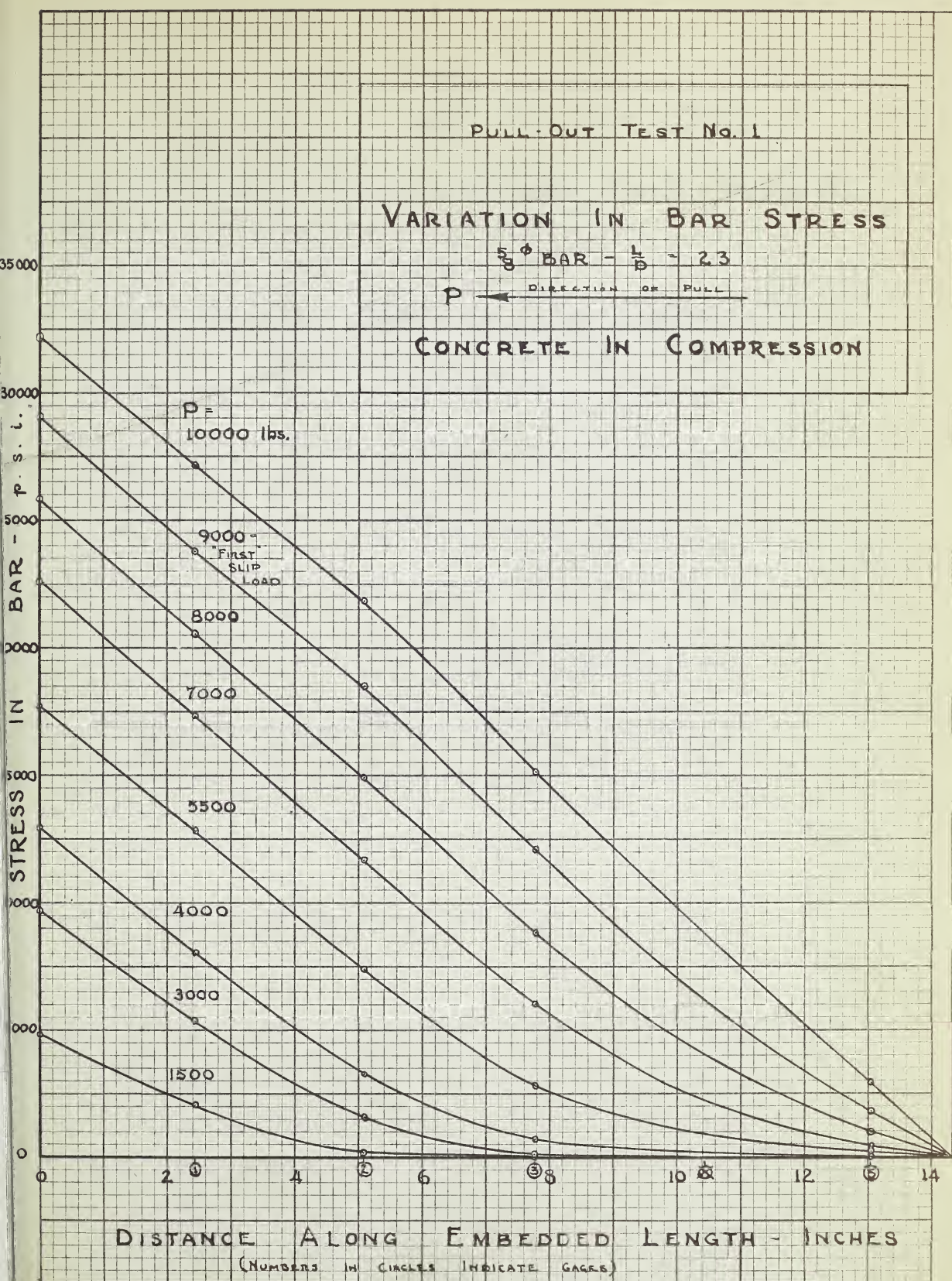
An article* in the February, 1949 Journal of the American Concrete Institute gives a graph showing the steel stress distribution in a concrete pull-out specimen using SR-4 gages. The steel stress-distribution curves found in Test No 1 as described previously agree very well in general shape with the curves given in the above article. In this case, the curves shown were for a $\frac{3}{4}$ " $\square$ plain bar in an 8" x 16" cylinder.

* "Experimental Aids in Structural Concrete Design"
Glover, Olsen, and Tangar.

TABLE I

DETAILS OF PULL-OUT TESTS

PULL-OUT TEST TYPE	MIX DETAILS	AUX. STRENGTH TESTS 28 DAY STR.	TOTAL LENGTH OF EMBEDMENT	L/D RATIO	AVERAGE BOND STRENGTH AT FIRST SLIP
1.)  5/8" PLAIN BAR CONCRETE IN COMPRESSION	MAT'L'S DONCASTER SAND ALB. CONC. PRODUCTS 1" ROUNDED GRAVEL PROPORTIONS 1 : 3.18 : 3.80 : 0.65 SLUMP 3 1/2"	CYL. NO. 1. 2800 2. 2610	INS. 14.3	23	p.s.i. 320
2a.)  5/8" PLAIN BAR CONC. IN COMP.	ELK ISLAND SAND 1" ANGULAR GRAVEL 1 : 3.6 : 3.4 : 0.67 4" SLUMP	(20 DAY STR. GIVEN) 10. 2610 11. 2340 12. 2230	7.5	12	350
2b.)  5/8" BAR NO GAGES	SAME AS 2a.	AS FOR 2a.	7.5	12	300
3.)  5/8" BAR CONCRETE IN TENSION	DONC. SAND A.C.P. 1" ROUNDED GRAVEL 1 : 3.18 : 3.80 : 0.65 4 1/2" SLUMP	3. 2770 4. 2890 5. 2780 6. 2900	13.8	22	? (FIRST SLIP NOT MEASURED)
AUXILIARY TO BEAM TEST  5/8" BAR NO GAGES	SAME AS 2a.	(30 DAY STRENGTH) 7. 2500 8. 3540 9. 3360 BOND CYL. 2825	11.9	19	336



BOND STRESS - p.s.i.

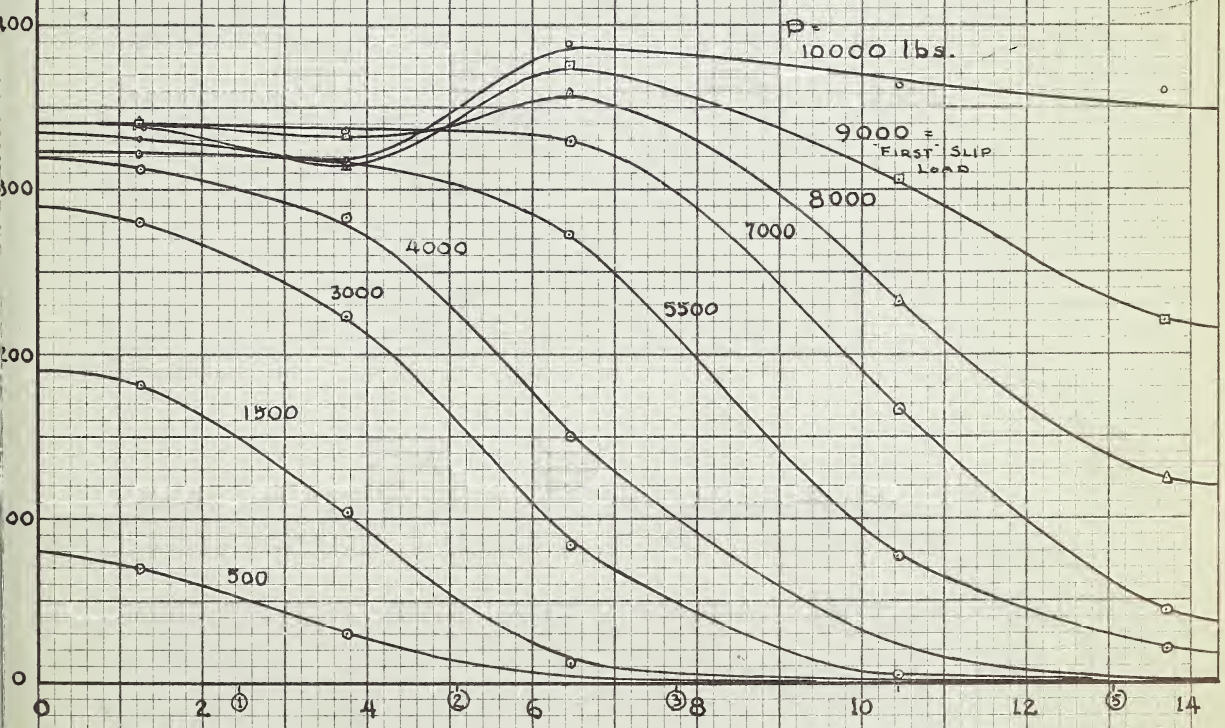
PULL-OUT TEST NO. 1

VARIATION IN BOND STRESS

$\frac{5}{8}$ " BAR - $\frac{L}{D} = 23$

$P \leftarrow$

CONCRETE IN COMPRESSION



DISTANCE ALONG EMBEDDED LENGTH - INCHES

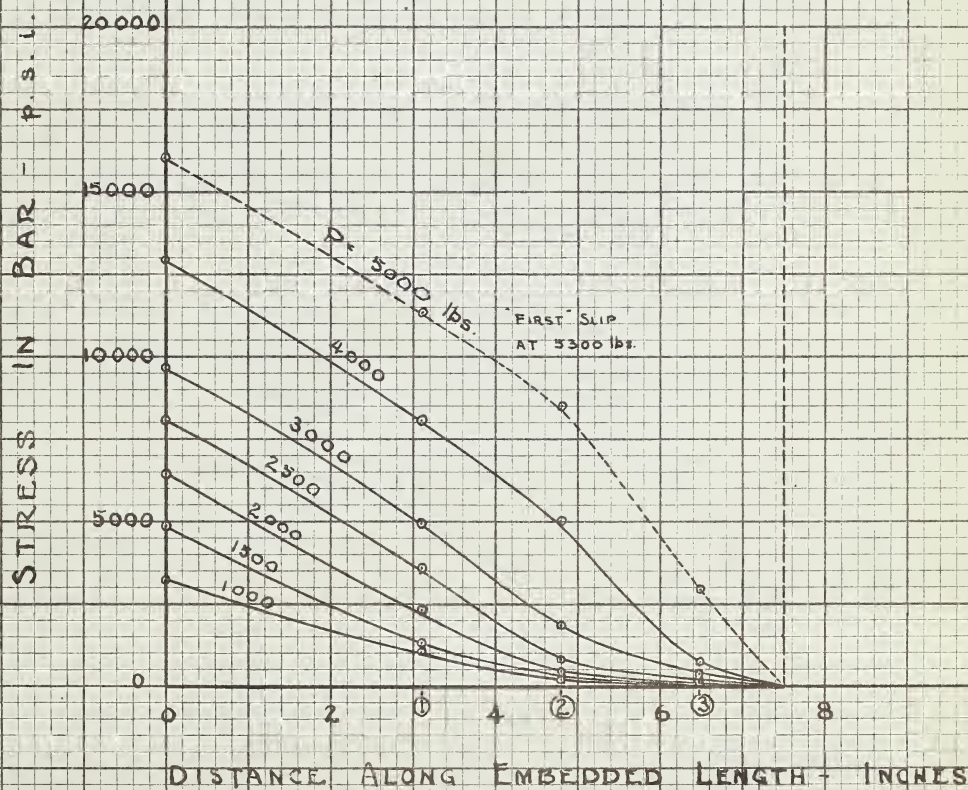
PULL-OUT TEST No. 2

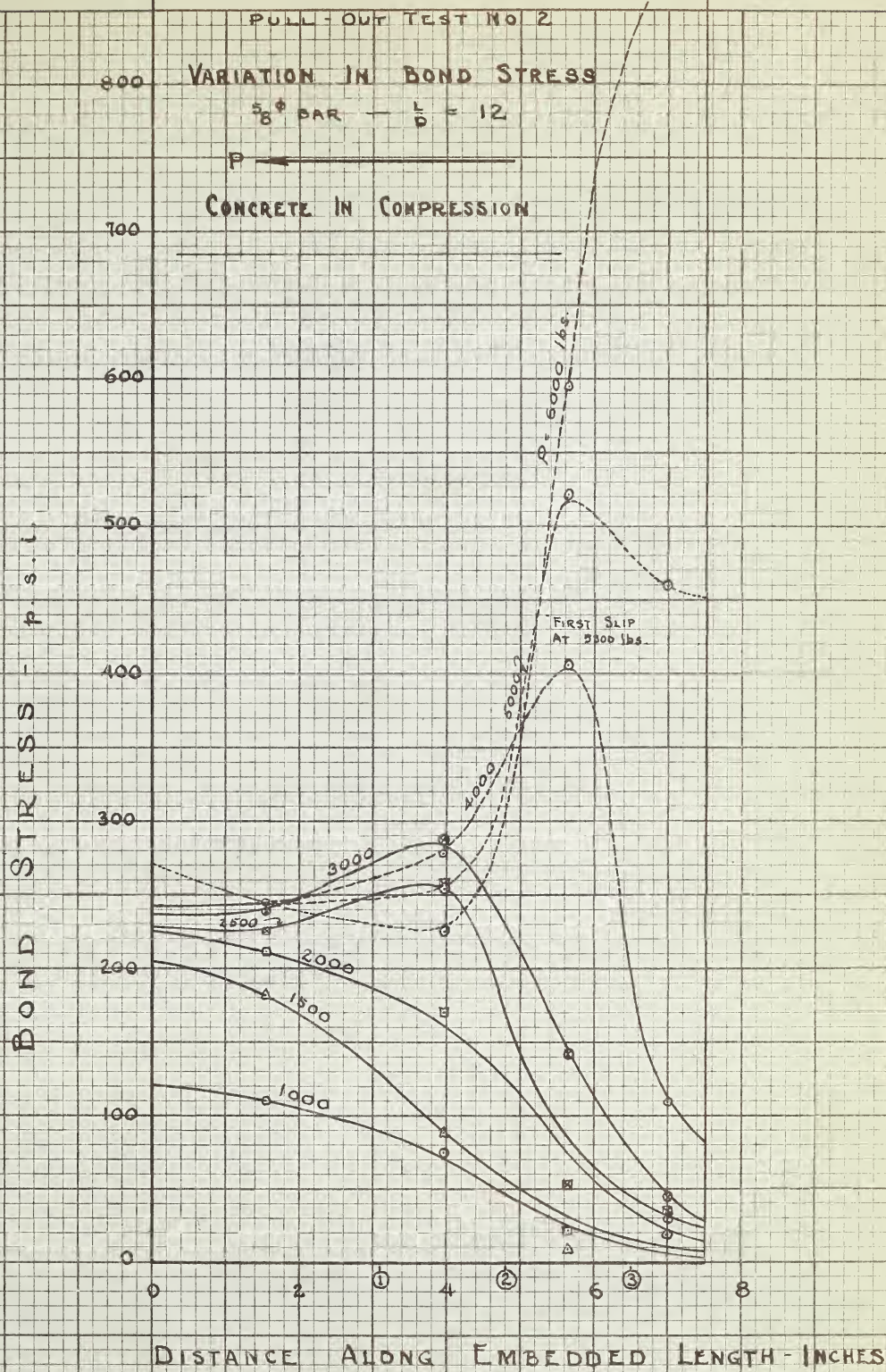
VARIATION IN BAR STRESS

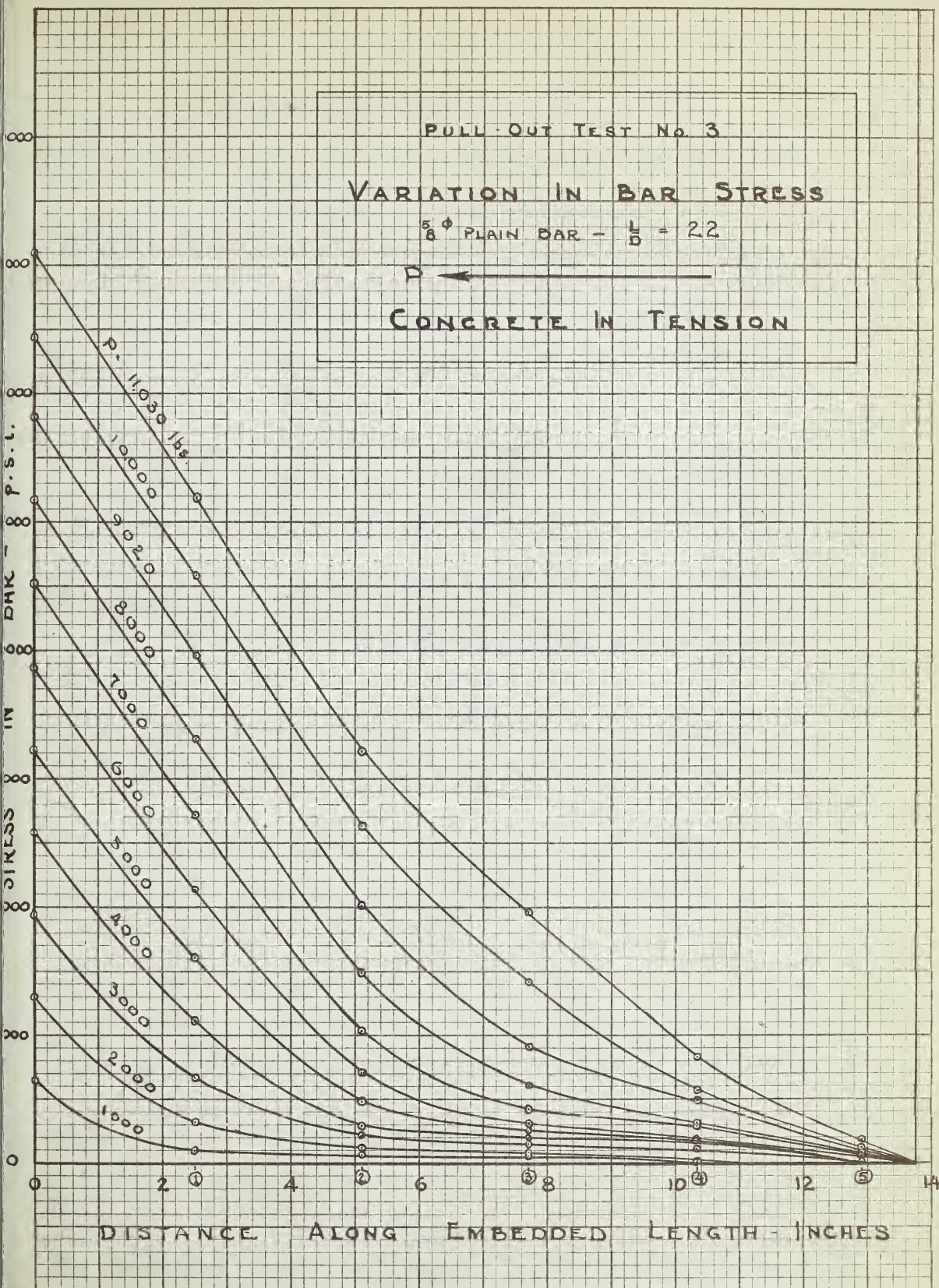
$$\frac{5}{8} \phi \text{ BAR} - \frac{L}{D} = 12$$

P ← —————

CONCRETE IN COMPRESSION







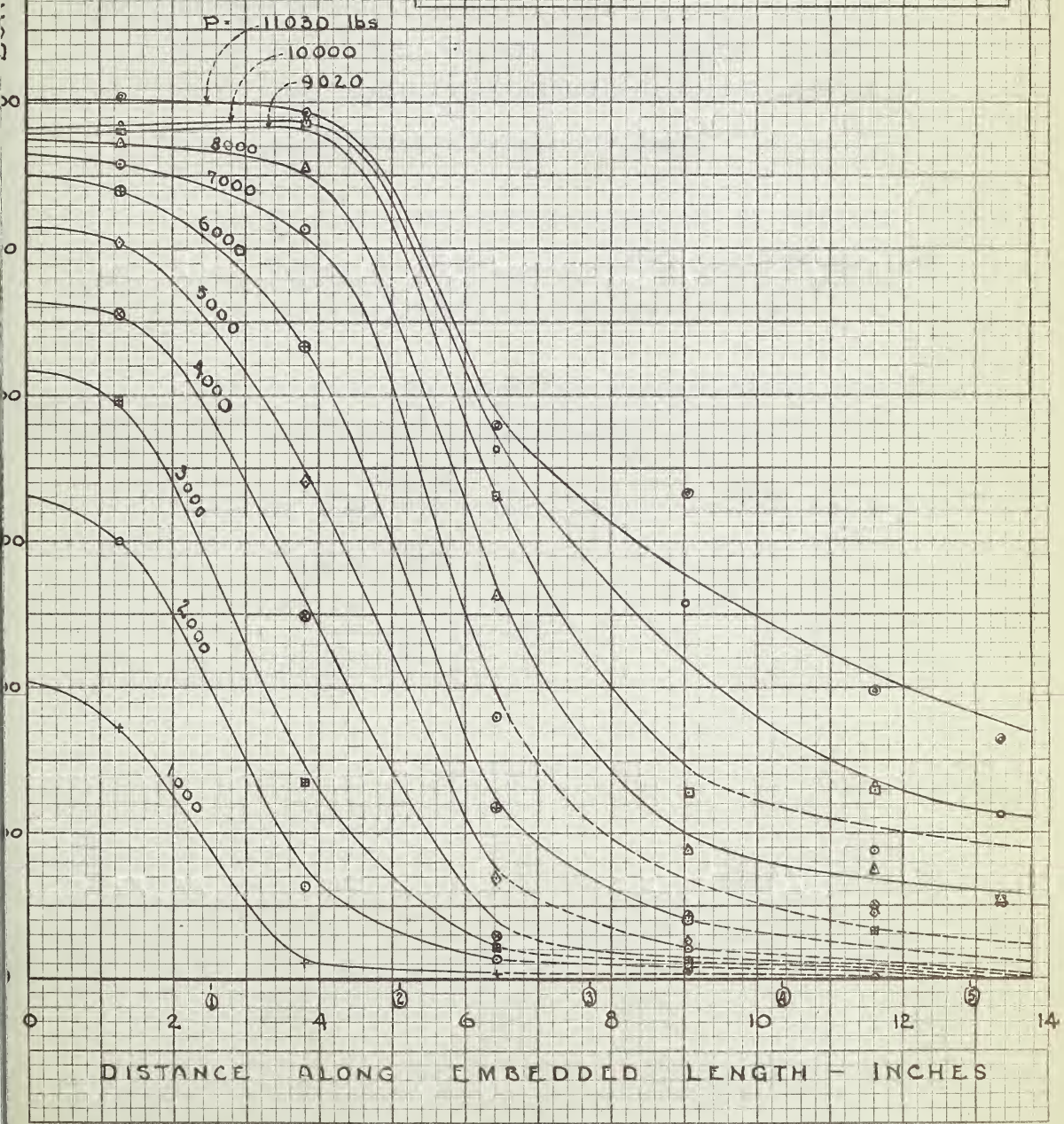
PULL-OUT TEST NO. 3

VARIATION IN BOND STRESS

5 ϕ PLAIN BAR - $\frac{L}{D} = 22$

P ← —————

CONCRETE IN TENSION



VI.

BEAM TEST

As part of this work on the study of bond a two-span continuous reinforced concrete beam was designed and tested.

(a) Purpose

The objectives to be studied in the beam test were as follows:

- (1) To measure the intensity of direct stress in both tensile and compressive reinforcement at various points along the bars.
- (2) To obtain the bond stresses due to shearing forces from the above stress measurements, and to compare them with the theoretical.
- (3) To study the anchorage of reinforcing bars in tension and in compression concrete.
- (4) To study the value of hooks in anchoring reinforcing bars.

(b) Design of the Beam

The actual support conditions of the beam as it was to be tested were kept in mind in designing the reinforcing. The beam was to be made up of two five-foot spans with a single concentrated load at the mid-point of each span. It would be supported at its ends on hinged supports resting on a double 12" at 40.8 lb. I-Beam placed across the table of the testing machine. The reaction at its mid-point would be supplied by a 200,000 lb. capacity proving ring. This meant that the beam would be supported by a relatively rigid centre support, and two equally-sinking end-supports. A calculation was made to find out the increase in negative moment over the centre support caused by this condition.

The computation for the effect of the sinking end supports was carried out on the basis that the proving ring provided a completely rigid support. By equating the equations for the downward deflection of the concrete beam (with assumed reinforcing) and of the double I-beam, the value of the unknown end reaction was found. This value turned out very close to $7/8$ of the end reaction for a non-sinking end support. The diagram on the next page gives the theoretical Shearing Force and Bending Moment diagrams which were used in the design. It may here be pointed out that the values for the maximum shear and for negative moment at mid-span for the type of loading shown are $0.6875 P$ and $0.1875 P l$ respectively if the supports are rigid throughout.

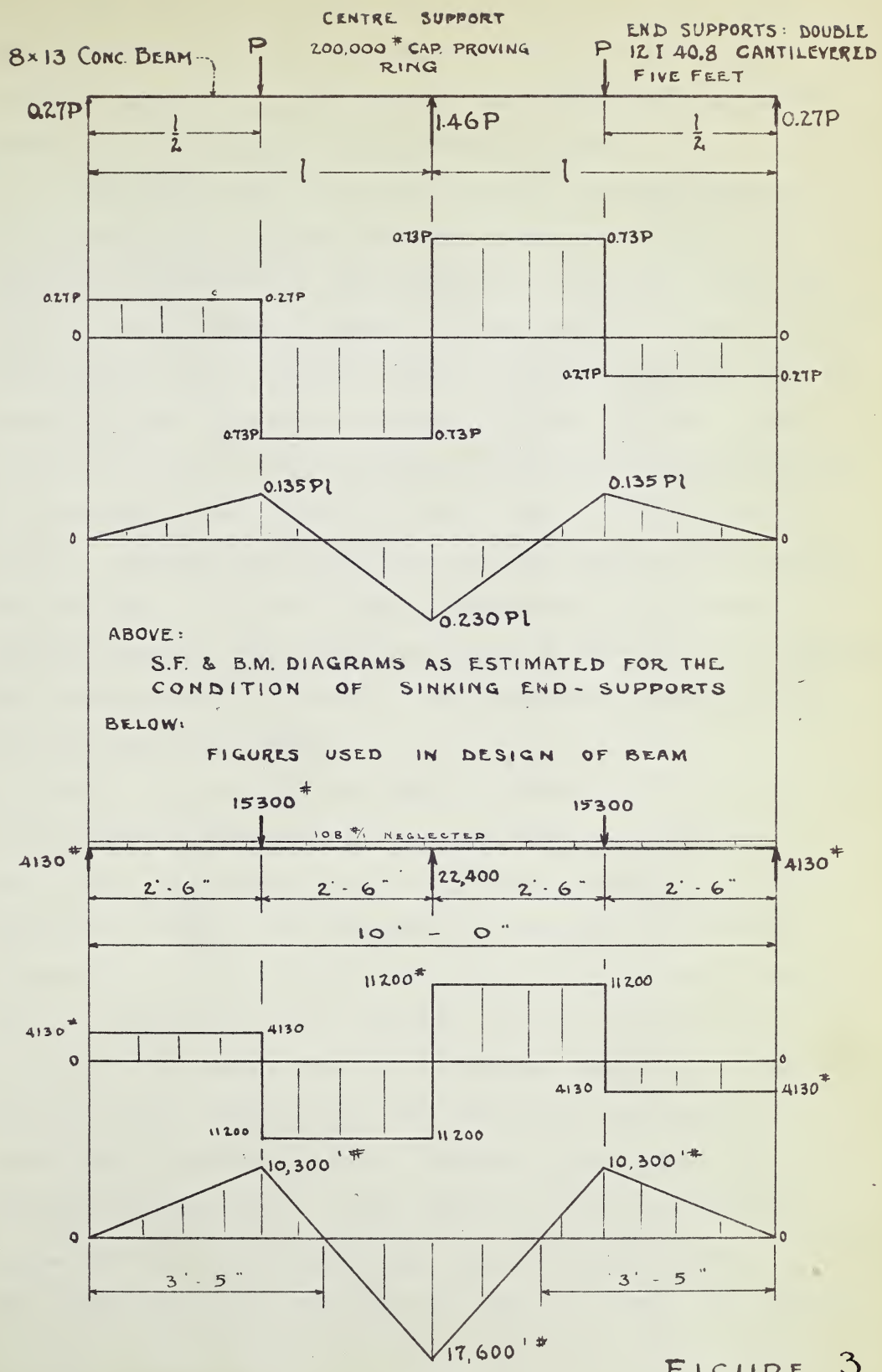


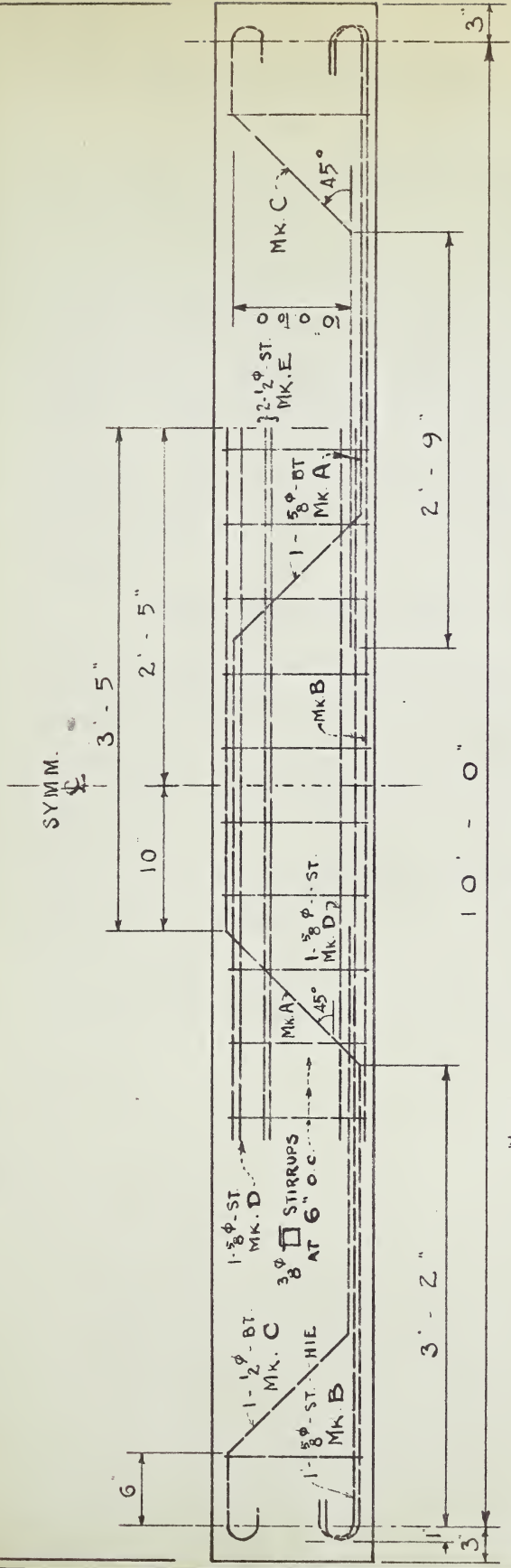
FIGURE 3.

The beam was designed according to the U.M.S.A. Code on Reinforced Concrete, for the chief reason than an idea might be gained as to the value of some of the code requirements. A brief summary of the design follows:

The design load "P" on the 8" x 15" beam was figured on the basis of a maximum shearing stress of 150 p.s.i. in the beam 2500 lb. concrete. This load came to 15,300 pounds, and gave a maximum negative moment over the centre support of 17,600 foot-pounds, and a positive moment of 10,300 foot-pounds at the quarter-point. The steel required for this positive moment consisted of 2- $\frac{5}{8}\phi$ and 1- $\frac{1}{2}\phi$ bars. The tensile reinforcing over the mid-support was made up of 3- $\frac{5}{8}\phi$ and 2- $\frac{1}{2}\phi$ bars in two layers. Compressive reinforcing was also required at mid-span and was made up of 3- $\frac{5}{8}\phi$ bars. Stirrups at 6" o.c. were required between load points, and were in the form of $\frac{3}{8}\phi$ rectangular closed hoops. A check on the theoretical bond stresses gave the following values:

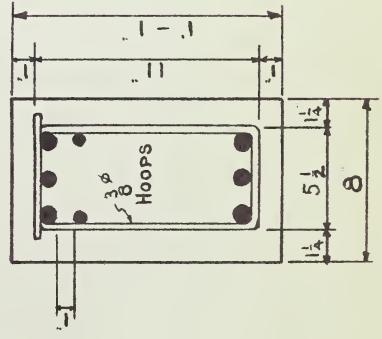
on tensile bars over the support, a maximum of 140 p.s.i.;
on the tensile bars near the quarter point, a maximum of 222 p.s.i. This latter value slightly exceeds the U.M.S.A. allowable of 208 p.s.i. for 2500 lb. concrete. The length of anchorage of bars beyond the section of zero Bending moment was computed to be as follows, according to the Code: A total of 7.5 inches for the top tensile bars; 8.7 inches for the bottom tensile bars, and nil for the compression bars over the centre support. As strain gages would use up some anchorage area, it was decided to run all bars 10 inches past the point where they were no longer needed.

The design details of the beam are shown in Figure 4, page 31.



NOTE:

PLAIN ROUND BARS THROUGHOUT
INTERMEDIATE GRADE STEEL
2500 # CONC. - 1" MAX. AGGREGATE



SECTION AT C
SCALE: 1 1/2" = 1'-0"

BEAM DESIGN DETAILS

SCALE: 1" = 1'-0"

FIGURE 4

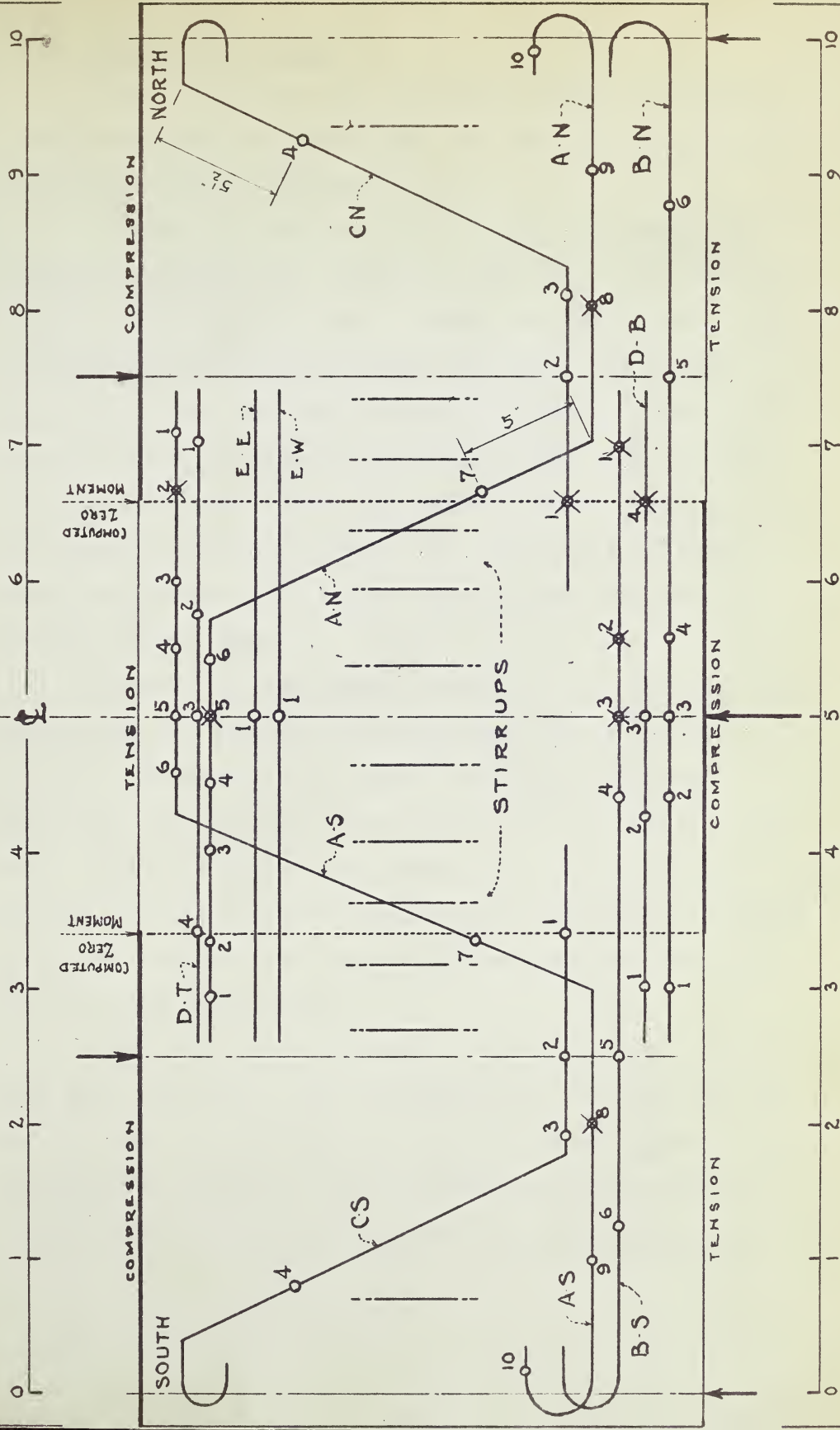
(c) Location of Strain Gages

Figure 5 shows the location of the gages as poured in the beam. The vertical dimension of the drawing has been distorted in order to avoid crowding. The identification lettering is also given for the various bars. In general this differentiates the five types of bars, A, B, C, D, and E as to North and South, Top and Bottom, East and West. This identification was later used in reading the gages. Thus a gage numbered as C-S 3 meant the third gage on the C-type bar located in the South span of the beam.

Fifty gages were used in all. As may be seen from the drawing, this did not allow a very close spacing of gages on the bars if the stresses were to be measured throughout the beam. It was thought necessary to place the gages symmetrical to the centre-line of the beam so that a check on each reading could be obtained. In order to eliminate unknown bending effects in the bars the gages were placed on the side of the bar at mid-height. The strains measured would then be near the average in the bar.

LOCATION OF GAGES

41 READ OUT OF 50



HORIZONTAL SCALE : 1" = 1'-0"
VERTICAL SCALE DISTORTED

FIGURE 5

(d) Pouring and Testing

Prior to pouring all gages were tested for leakage resistance and true resistance. At this time three faulty gages were found (See data).

The beam was poured using a 1:3.6 :3.4 :0.6 .7 mix designed for 2500 lxb. concrete. The F.M. of the Sand was 3.48 and the gravel was 1" angular and round mixed. The concrete was well rodded around the bars. Due to the stirrups having been made somewhat oversize, it was necessary to add 0.8 inch to the depth of the form in order to get sufficient cover. The poured size of the beam was thus 8" x 15.8" x 10'-6". All electrical lead-out wires (2 per gage in a single conduit) were brought out the top of the beam.

After 26 days of curing under moist sand the beam was made ready for testing. Six gages had spoiled due to moisture leakage into the gage, leaving 41 out of 50 available for readings. These gages were connected to the SR-4 Strain Indicator via a switch-box, all electrical connections being soldered, and wrapped with tape. A "dummy" gage, for temperature change compensation, was attached to the beam during the test.

Figure 6 shows a photograph of the proving ring as set under the beam. The end supports of the beam were set at the same elevation as the top-plate of the proving ring. The beam was placed on these supports and then jacked up slightly near the centre to allow the proving ring to be

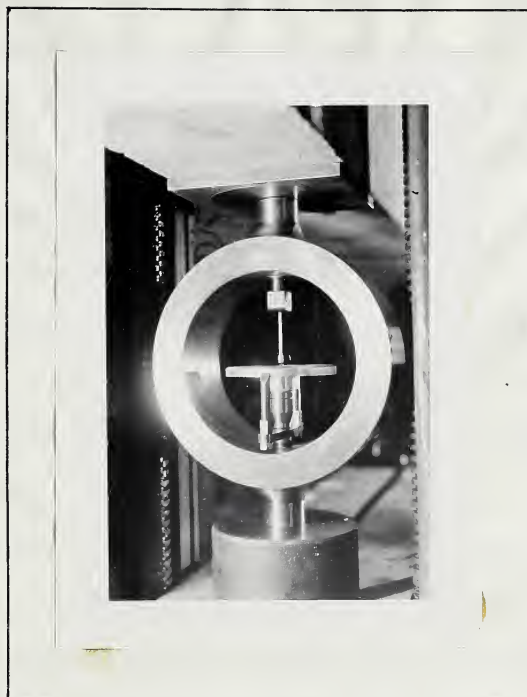


FIGURE 6

PROVING RING ACTING AS
CENTRE SUPPORT OF BEAM

slipped under it. On releasing the jack-pressure it was found that the proving ring carried slightly over half the dead weight of the beam.

During the test the beam was loaded in increments of about 2000 lbs. applied at the quarter-points i.e. twice this load or 4000 lb increments on the testing machine. One cyclic loading was carried out, from 0 to 8000 to 0 pounds for "P" and then the loading was increased continually until failure. At least 10 minutes were allowed to elapse after each loading before readings of steel strain were taken. It was noticed by checking some gages twice for a given load that the steel strain increased slightly during the 5 to 15 minutes required to read all the gages; some of these double readings were recorded but the first ones taken were used in the computations.

In addition to the gages, readings were taken on the proving ring, and of the temperature at each load. Finally, the deflection of the beam at the end supports, and at the load points, was measured at each load using a precise tilting level reading to a hundredth of an inch.

During the last day of the test three auxilliary strength test cylinders were broken and an ordinary pull-out test on a plain $5/8$ " ϕ bar embedded in a standard 6 x 12 cylinder was performed.

(c) Results

The original data for the beam test and auxiliary tests may be found in Appendix B of this report.

Before studying the plot of the results, an idea of the value of the gage readings themselves may be gained from computation sheets Nos. 70 to 73. It may be seen that though not always agreeing closely in magnitude, corresponding gages almost invariably register the same kind of strain, and increase or decrease together.

After converting strain to stress by multiplying by 30×10^6 p.s.i., the strain gage data was plotted to give two types of graph. (See graphs that follow)

The first type, the Load-Stress diagram, shows the comparison between actual and theoretical steel stresses for all loads at any one section of the beam.

The second type, the Stress-Distribution diagram, shows the comparison between actual and theoretical steel stress over the full length of the bars at any one load. The value of the actual bond stress at various points along the bar may also be obtained from this type of graph.

The deflection readings taken were plotted to give the usual Load-Deflection curves for the concrete beam. In this case, due to the sinking end-supports, the true deflection of the beam at the load point had to be computed and this was done graphically.

Figure 7 shows several photographs taken of the beam test, showing various features of failure.



BEAM TEST



VIEW SHOWING DEFLECTED FORM OF BEAM

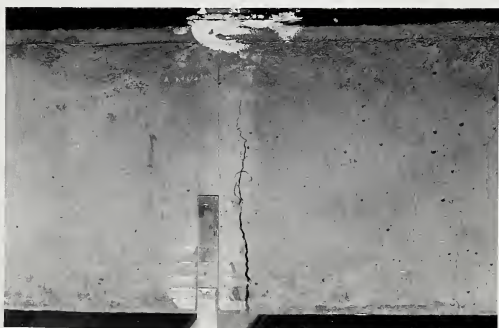
NOTE ARROWS



TENSION CRACK UNDER SOUTH LOAD



TENSION CRACK OVER PROVING RING



TENSION CRACK UNDER NORTH LOAD

(f) Interpretation

1. Load-Stress Curves.

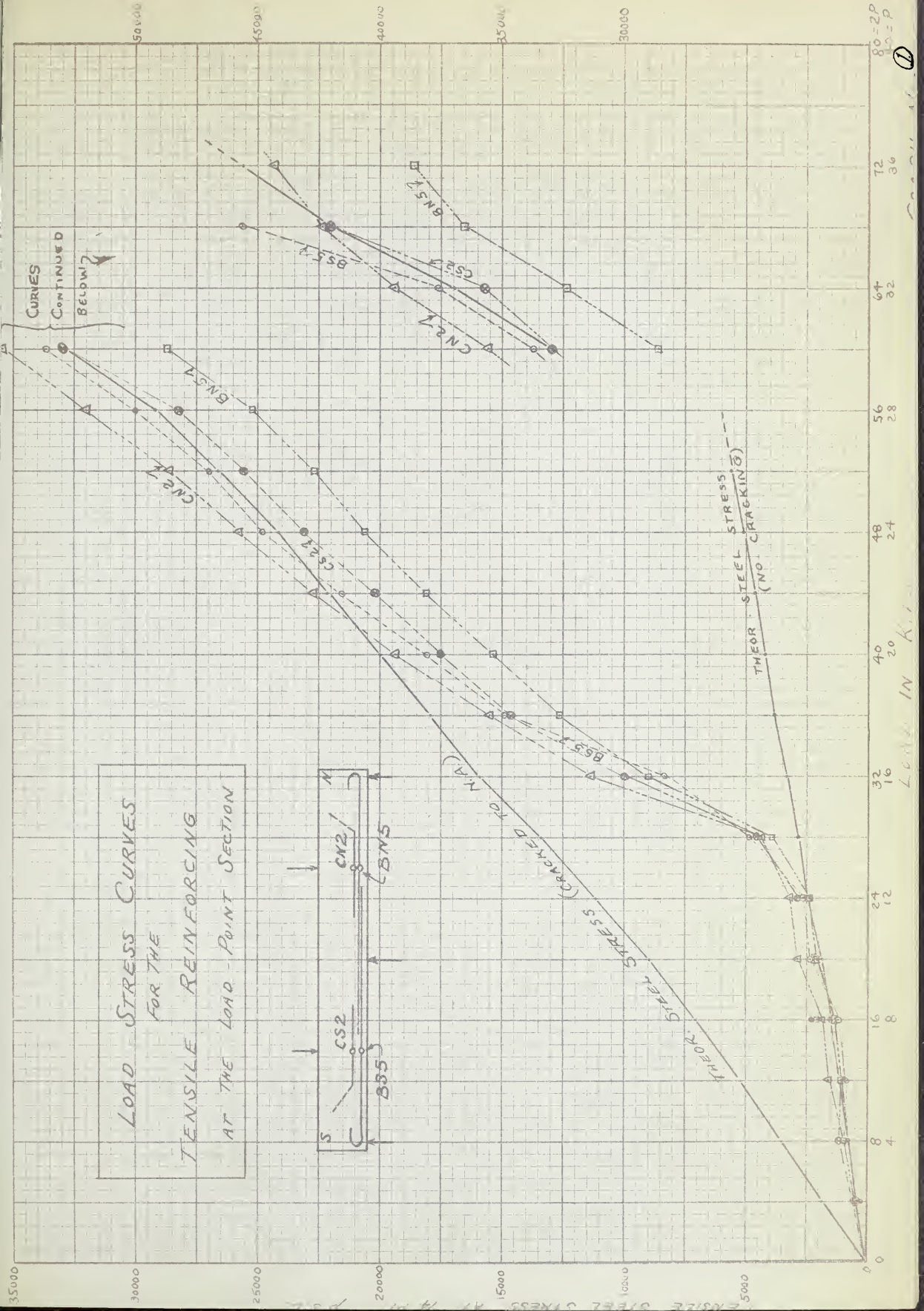
The Load-Stress curves give the best over-all picture of the variation of stress in the bars throughout the test. A study of these graphs shows the following features.

In general, the actual stresses vary in the same manner as the theoretical stresses, but usually differ considerably in magnitude from them. As a whole the steel stresses tended to be somewhat lower than the theoretical, showing that the concrete was doing more work than assumed. This is especially true at loads up to the design load (about 16,000 pounds for P), since little cracking had taken place in the concrete up to that time. At high loads, with much cracking, the actual steel stresses often exceeded the theoretical. Another general feature is that different bars at one section often showed large differences in stress carried.

The details of the various graphs are as follows:

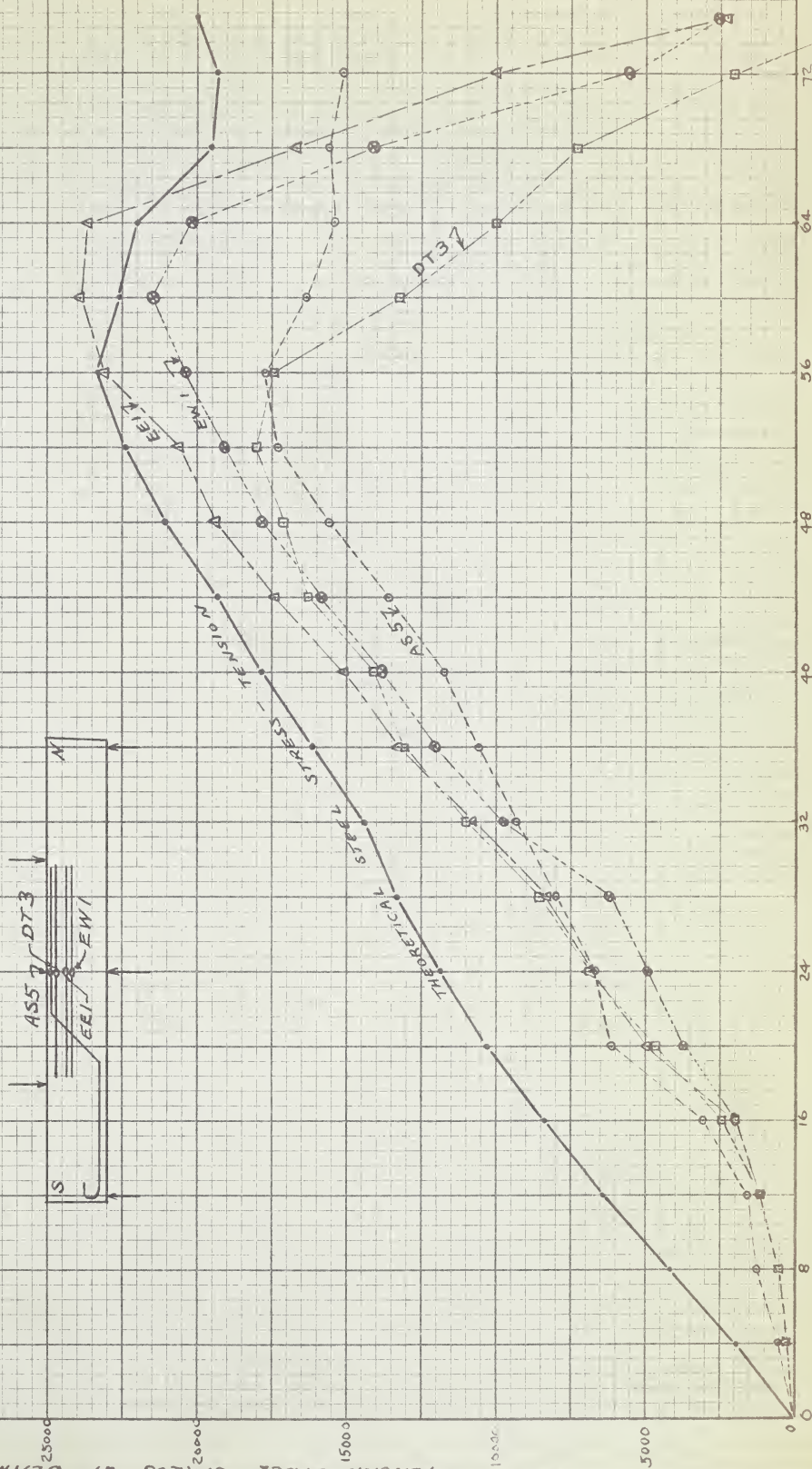
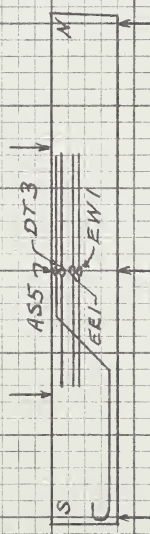
Graph No 1 shows good agreement between actual and theoretical tensile steel stress at the load-point section. A logical discrepancy arises as the section changes from the uncracked state to state where it is cracked up to near the neutral axis. The computed tensile stress in the concrete at a load of $P = 12,000$ lbs. where the steel stress first begins to rise sharply, came to 540 p.s.i. This value is about the maximum tensile stress that the concrete (3000 lb.) could be expected to take. It may be noted that

LOAD STRESS CURVES
FOR THE
TENSILE REINFORCING
AT THE LOAD-POINT SECTION



80-28
80-36
72
36
64
32
56
28
48
24
40
20
32
16
24
12
8
4
0

LOAD-STRESS CURVES for TENSILE REINFORCING - TOP ϕ_1



LOAD on BEAM - KIPS

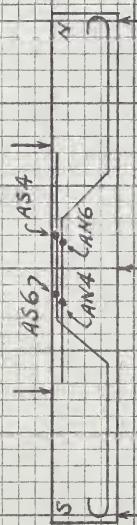
TENSION STRESS at Beam ϕ_1 - P.S.I.



LOAD - STRESS CURVES

for

TENSILE REINFORCING at TOP of
6" TO RIGHT and LEFT of ϕ .



25000

20000

15000

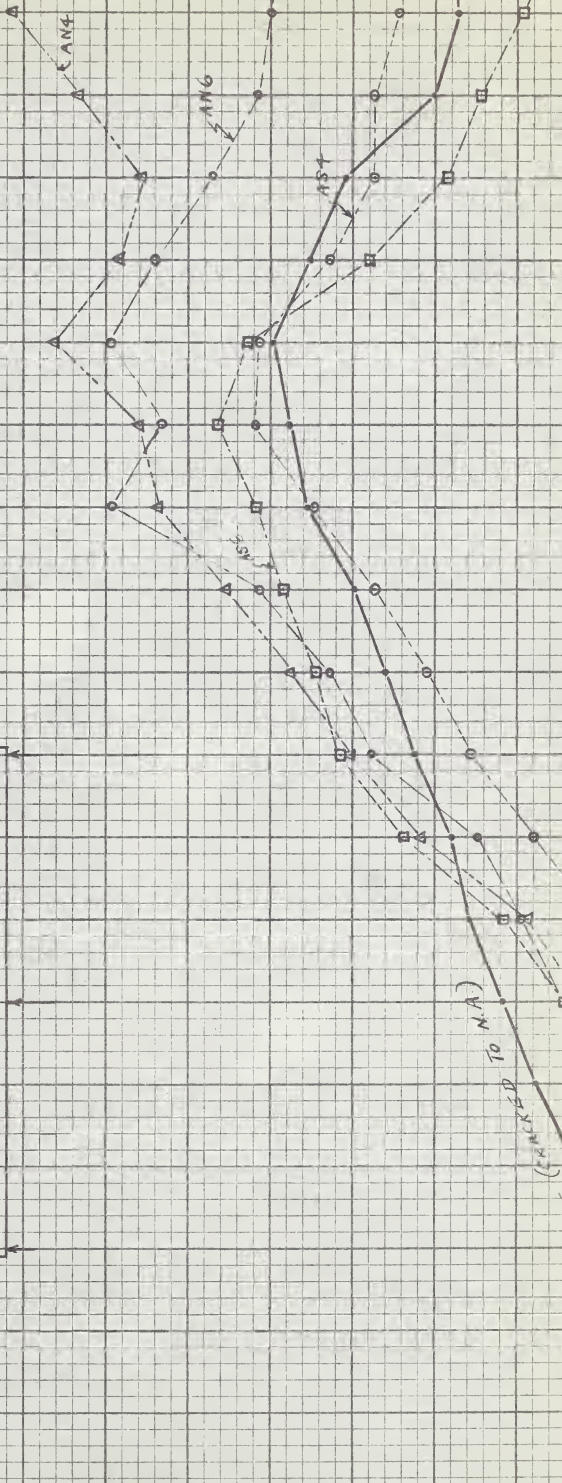
10000

5000

0

TENSILE STRESS - P.S.I.

STRESS (TRANSFERRED TO N.A.)

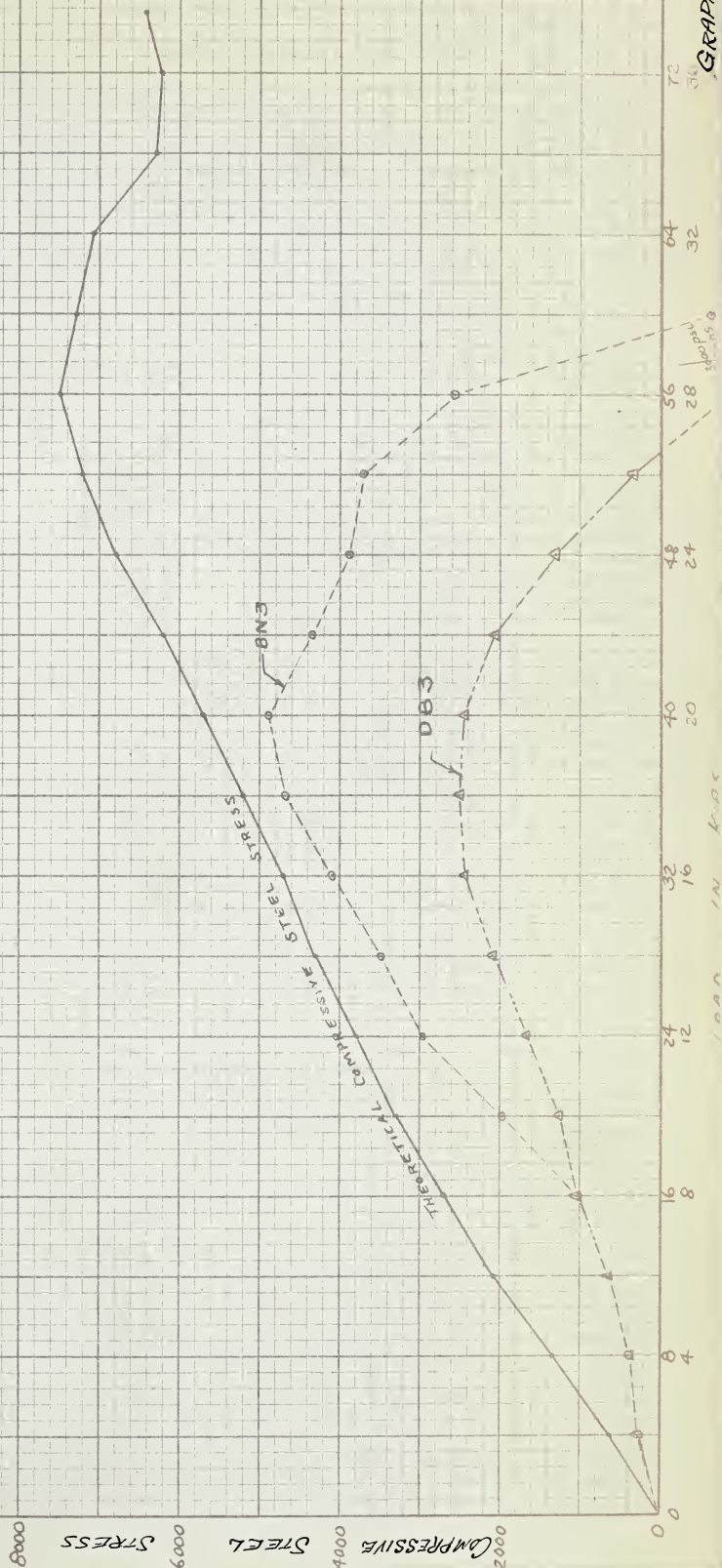
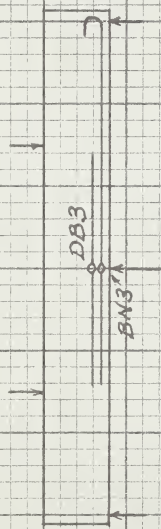


LOAD - STRESS CURVES

FOR

COMPRESSIVE REINFORCING - BOTTOM ϕ

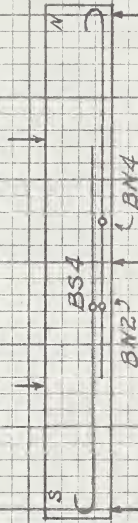
COMPRESSIVE
STEEL
STRESS
AT ϕ - P.S.C.



LOAD - STRESS CURVES

FOR

COMPRESSIVE REINFORCING AT BOTTOM
7" TO RIGHT AND LEFT OF C



THEORETICAL

BN2

BN4

BS4

LOAD P IN KIPS

12

8

4

0

2000

4000

6000

8000

10000

12000

14000

16000

18000

20000

22000

24000

26000

28000

30000

32000

34000

36000

38000

40000

42000

44000

46000

48000

50000

52000

54000

56000

58000

60000

62000

64000

66000

68000

70000

72000

74000

76000

78000

80000

82000

84000

86000

88000

90000

92000

94000

96000

98000

100000

0

2000

4000

6000

8000

10000

12000

14000

16000

18000

20000

22000

24000

26000

28000

30000

32000

34000

36000

38000

40000

42000

44000

46000

48000

50000

52000

54000

56000

58000

60000

62000

64000

66000

68000

70000

72000

74000

76000

78000

80000

82000

84000

86000

88000

90000

92000

94000

96000

0

2000

4000

6000

8000

10000

12000

14000

16000

18000

20000

22000

24000

26000

28000

30000

32000

34000

36000

38000

40000

42000

44000

46000

48000

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54000

56000

58000

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62000

64000

66000

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36000

38000

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56000

58000

60000

62000

64000

66000

68000

70000

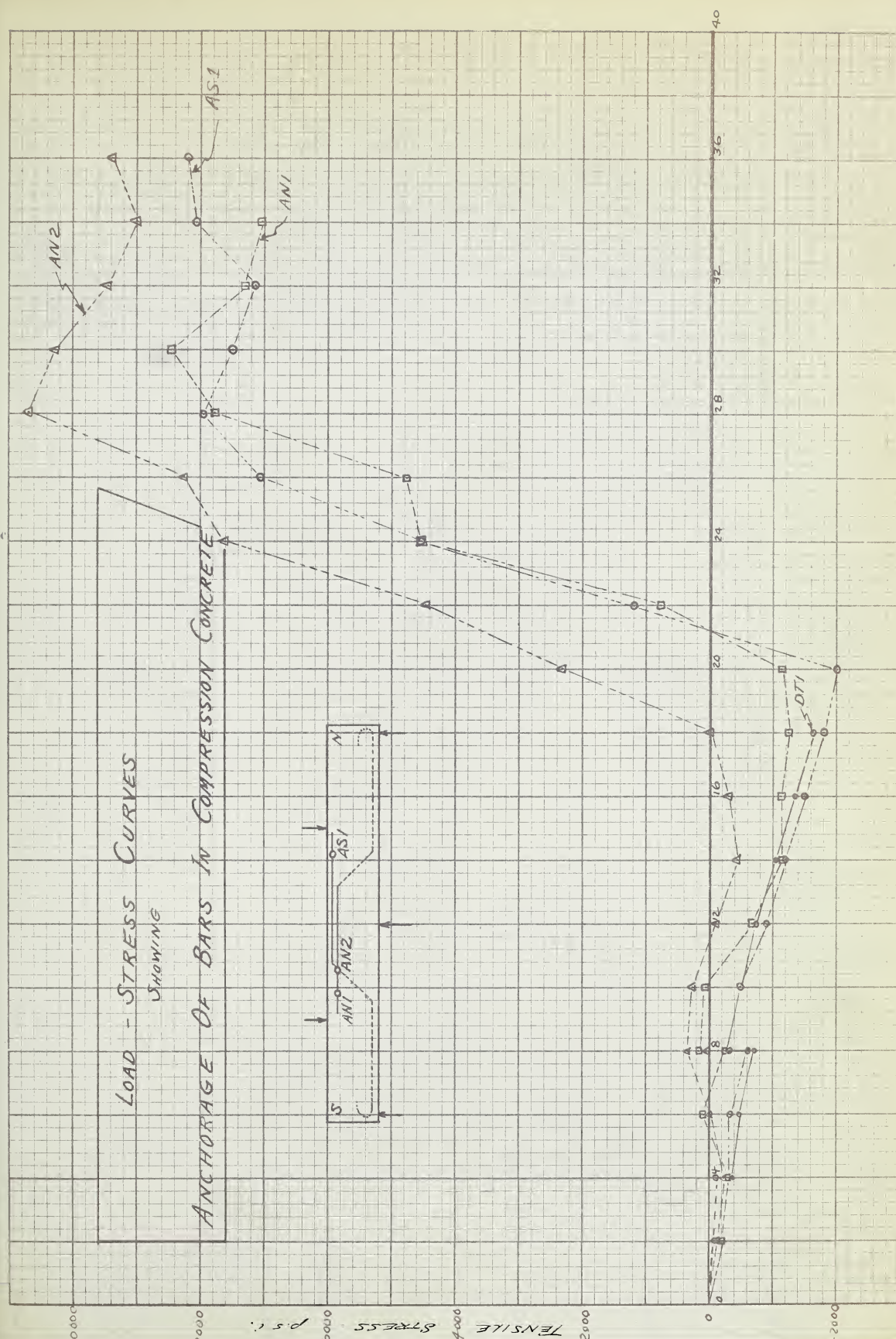
720

the theoretical steel stress lines are not straight because the proving ring took varying fractions of the total load. Also, for the graph shown those values of stress measured beyond 40,000 p.s.i. are in doubt as the yield point of the steel was at this value.

Graphs 2 and 3 show the tensile steel stresses measured in the top bars directly above the proving ring and at about 6" to the side of the $\mathcal{L}$ respectively. An important feature of Graph No 2 is that with a Load of P above 32^k , three of the top bars begin to lose stress very rapidly. In graph 3, the actual stresses at about 6" to right and left of $\mathcal{L}$ agree fairly closely with the theoretical.

Graphs 4 and 5 show the variation of stress in the compression steel at sections directly over the proving ring, and at 7" to each side of $\mathcal{L}$ respectively. The agreement with the theoretical stress is good at the latter sections, but there is a large difference at the $\mathcal{L}$ section. Graph 4 shows that the compressive steel over the support eventually carried tensile stresses. The change-over occurred at a load of P equal to about 28^k , or 1.75 times the design load, and the tensile stresses eventually reached some 23,000 p.s.i. (See computation sheets 74 and 76). This change over also occurred at 7" from the $\mathcal{L}$, but not until later when P had reached 32^k .

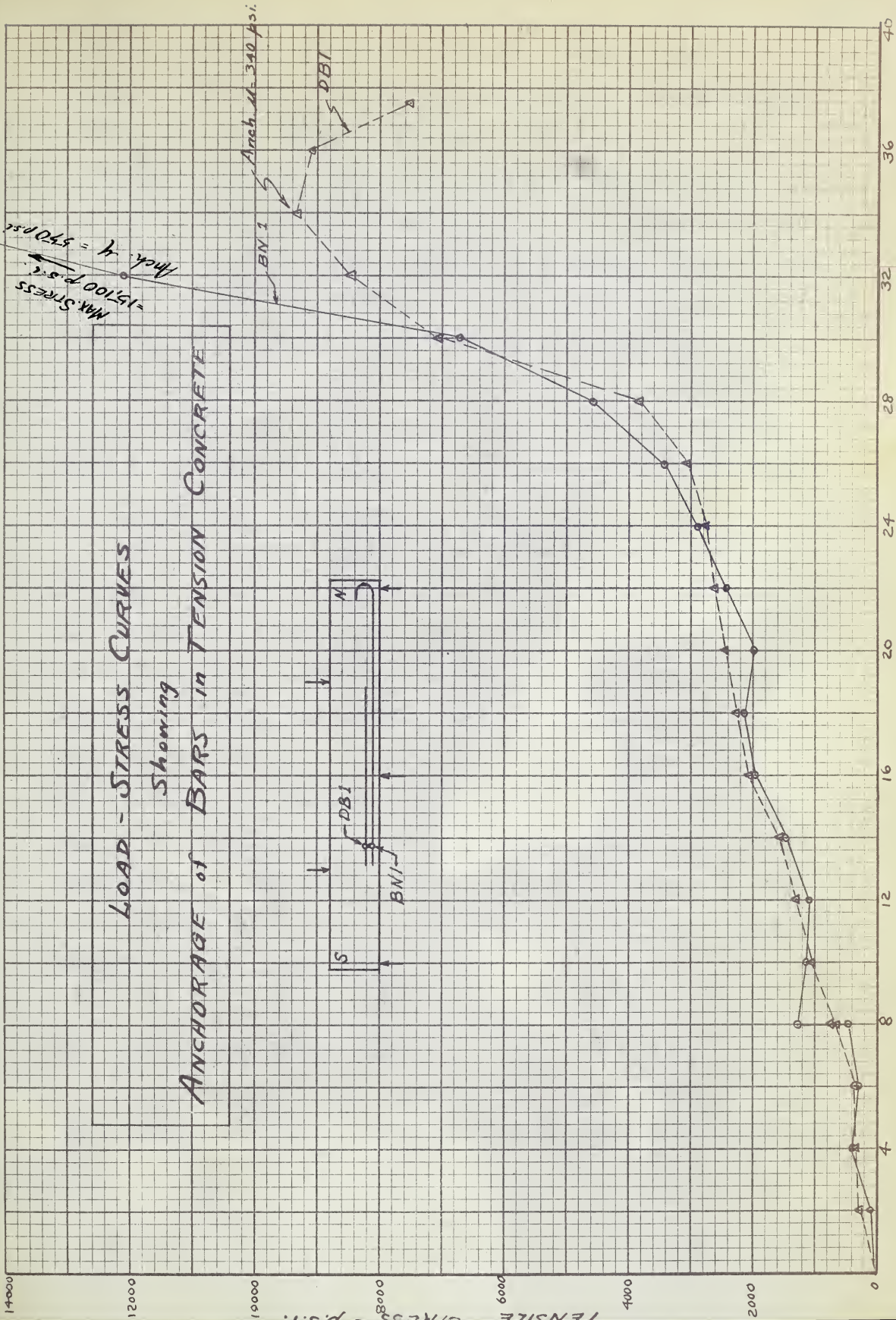
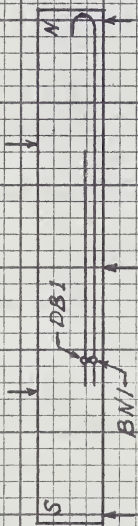
Graphs 6 and 7 give the stress intensities occurring in the anchored lengths of some of the bars. Graph 6 is



LOAD-STRESS CURVES

Showing

ANCHORAGE OF BARS IN TENSION CONCRETE



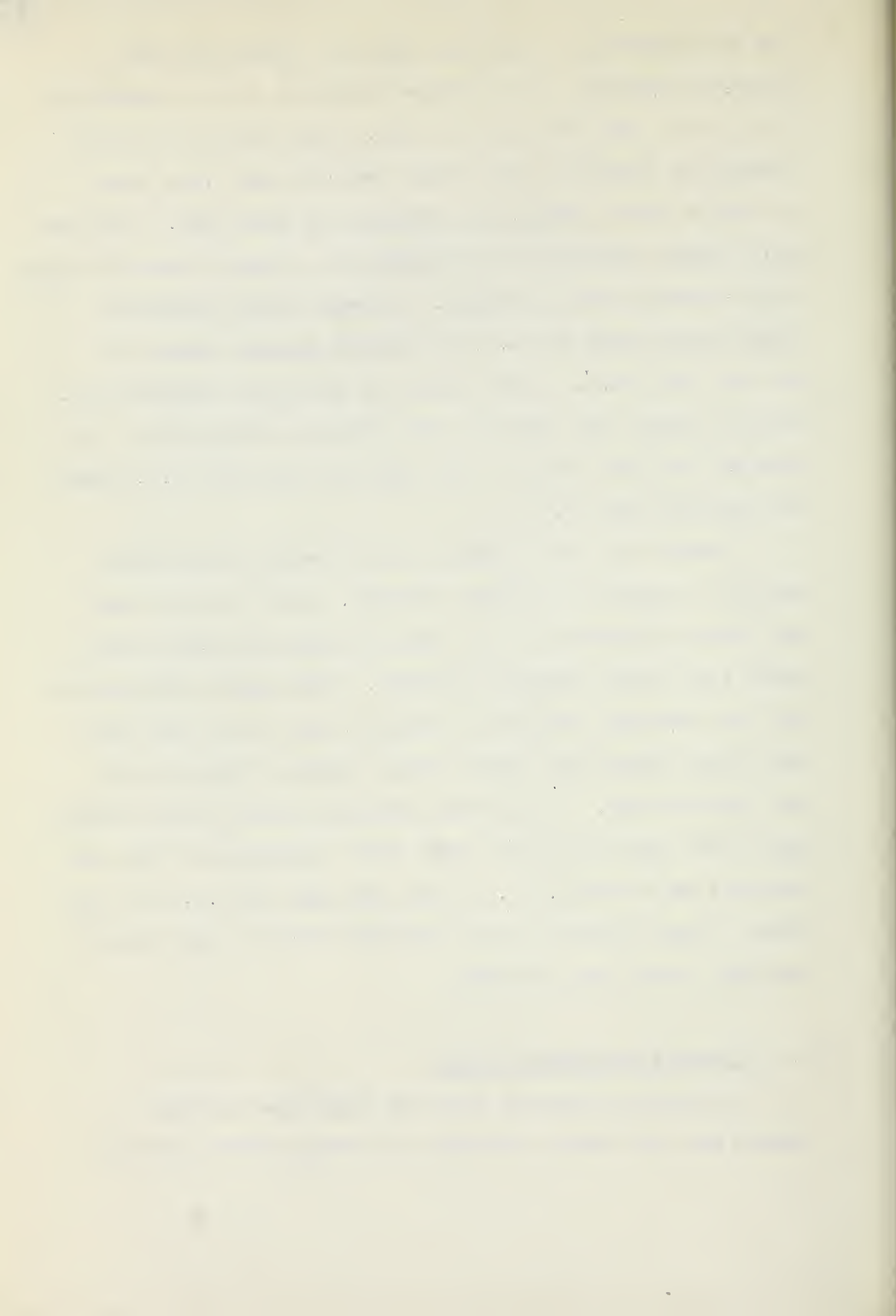
LOAD P IN KIIPS

for the anchorage of the top bars in a region of compression concrete. The anchored portions were in compression up to a load of about 20 kips, after which the stress changed to tension. This would indicate that they began to fulfil their purpose as anchorage at that time. The tensile stress increased to a maximum at a load of about 28 kips, after which it fell somewhat. A check on the anchorage bond stress along the A.N. bar showed maximum values of 370 and 400 p.s.i. at the 28 and 30 kip loads respectively. This is likely the limit of the adhesive resistance of the bond on the bar, being of the order of the 340 p.s.i. found in Pull-Out Test No. 1.

Graph 7 is for anchorage of two bottom compressive bars in a region of tension concrete. The two gages show low tensile stresses up to a load of about 28 kips, after which the stress increased rapidly. This again would indicate the anchorage becoming effective, and is at about the load where these bars showed large tension stresses over the proving ring. The maximum anchorage bond stress occurring at the ends of the two bars when P equalled 34 kips was computed to be 550 p.s.i. on one bar, and 340 p.s.i. on the other. These figures are of doubtful value as they depend entirely on one gage reading.

2. Stress-Distribution Curves

The next few graphs show the comparison between theoretical and actual stresses all along various bars at



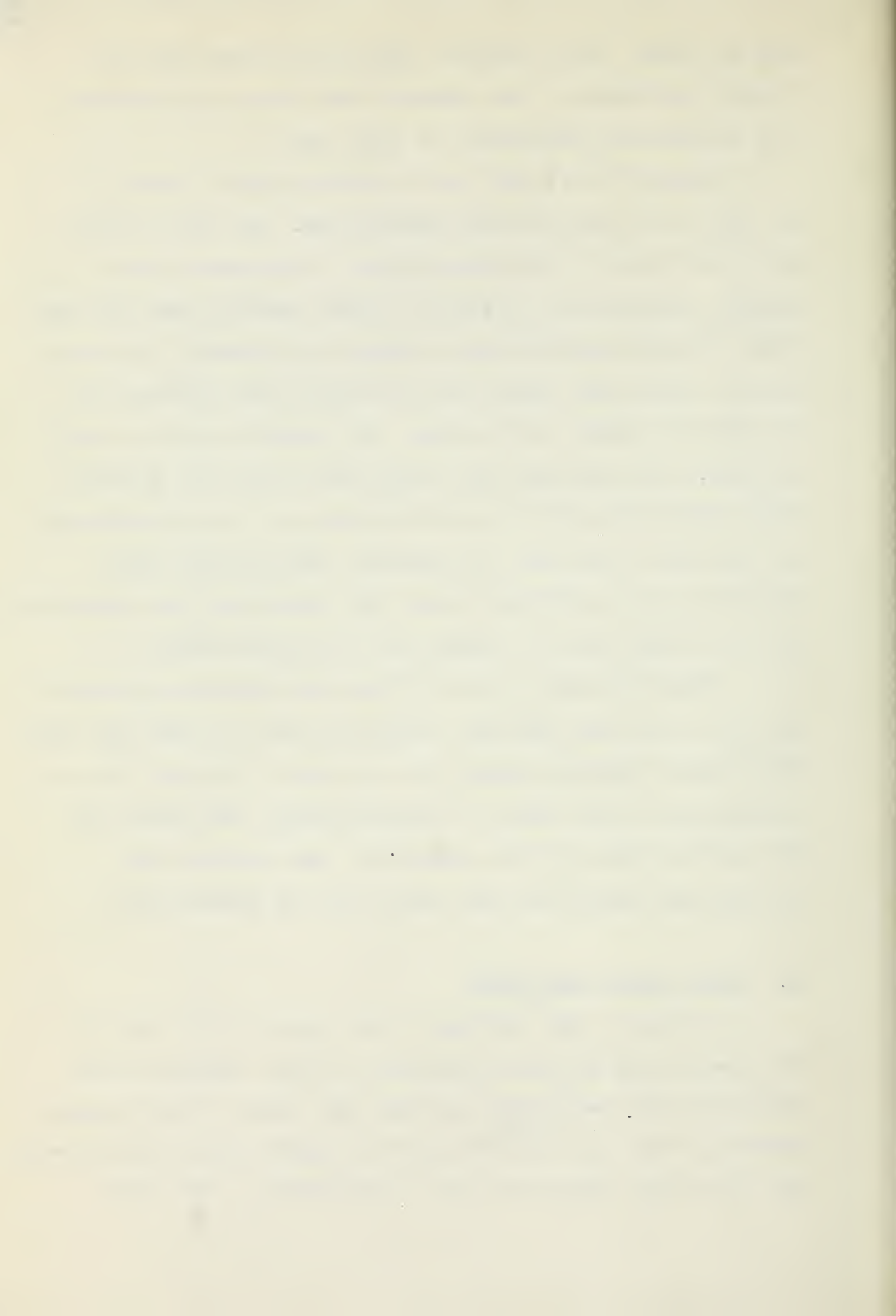
any one load. High values of load were chosen since the cracked condition of the concrete more nearly approximated the theoretical assumptions at that time.

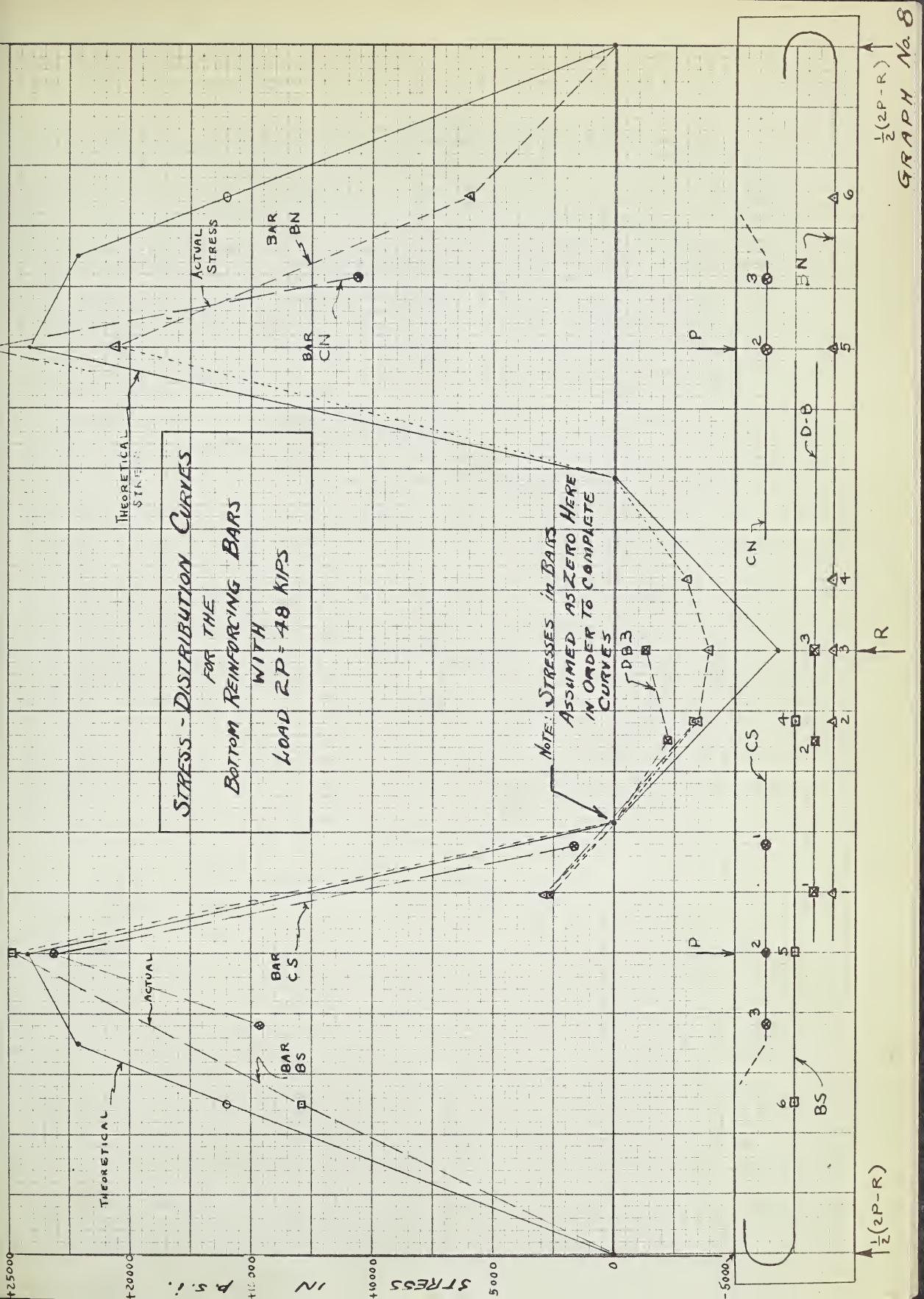
Graphs 8 and 9 are for the bottom bars at loads of $2P$ equal to 48 and 68 kips respectively. The latter load is in the order of the failure load. The curves show a general agreement as to type of stress, and at some sections there is good agreement as to values of intensity of stress. Graph 9 gives some values of the actual bond stresses due to shearing forces in the beam. The highest value is about 345 p.s.i. for the B-3 bar. There was no failure of bond under this stress but this was probably due to the anchorage of the hook on the bar. In general, the existing bond stresses due to shearing forces were lower than the theoretical as may be seen from the slopes of the stress curves.

Graphs 10 and 11 are the Stress Distribution graphs for the bars that were bent up, as well as for the other top bars. The tensile stresses across the top tend to be lower than the theoretical at the point of maximum moment, and higher as the distance from the $\mathcal{C}$ increases. Bond stresses due to shearing forces are much lower than the theoretical.

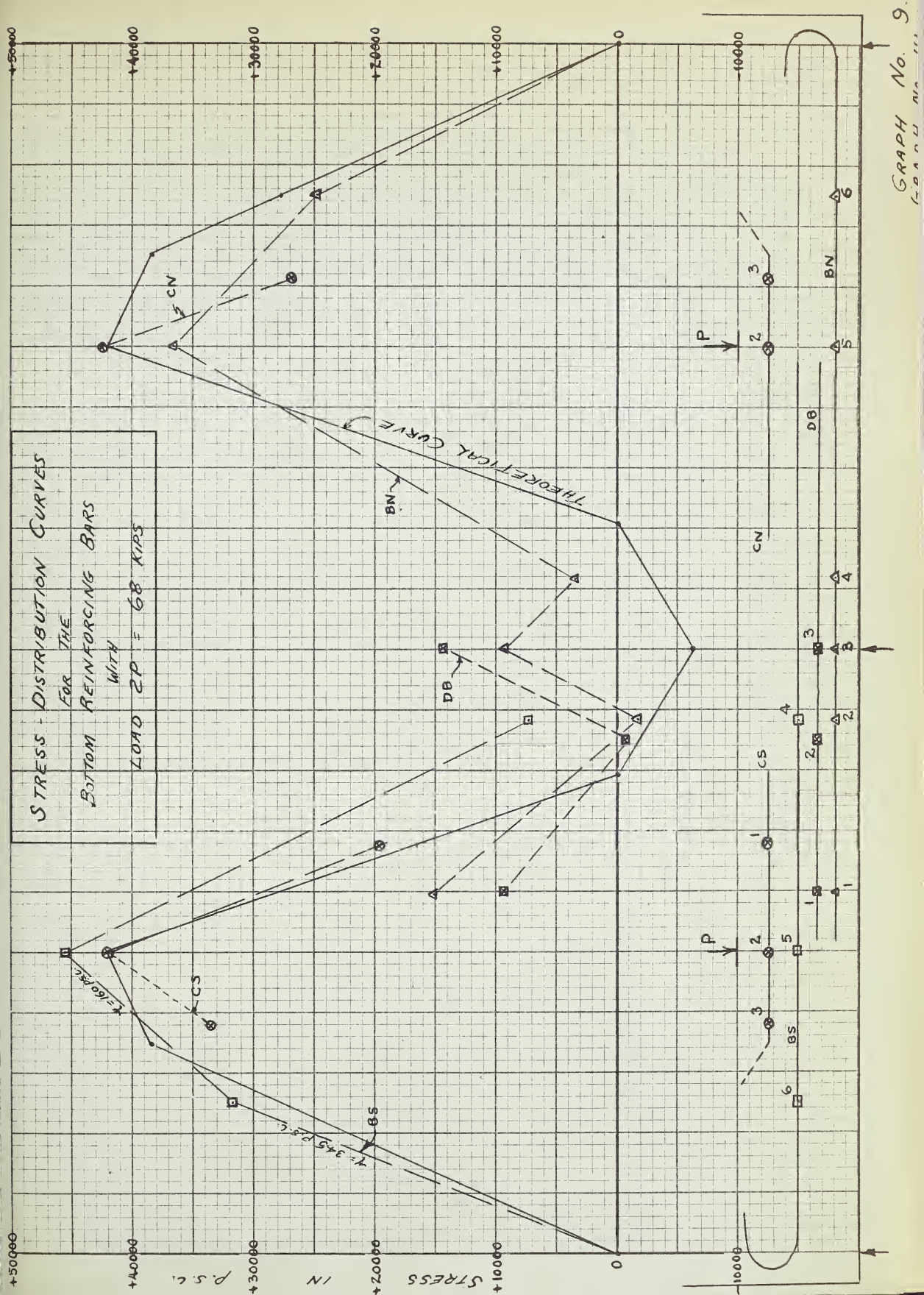
3. Other Strain Gage Data

A study of the readings of the gages on the ends of the hooks at AS 10 and AN 10 shows a slight tensile stress developing and increasing steadily but slowly at high loads. Readings taken up to a total load of 56,000 pounds were erratic but thereafter increased quite consistently. This would

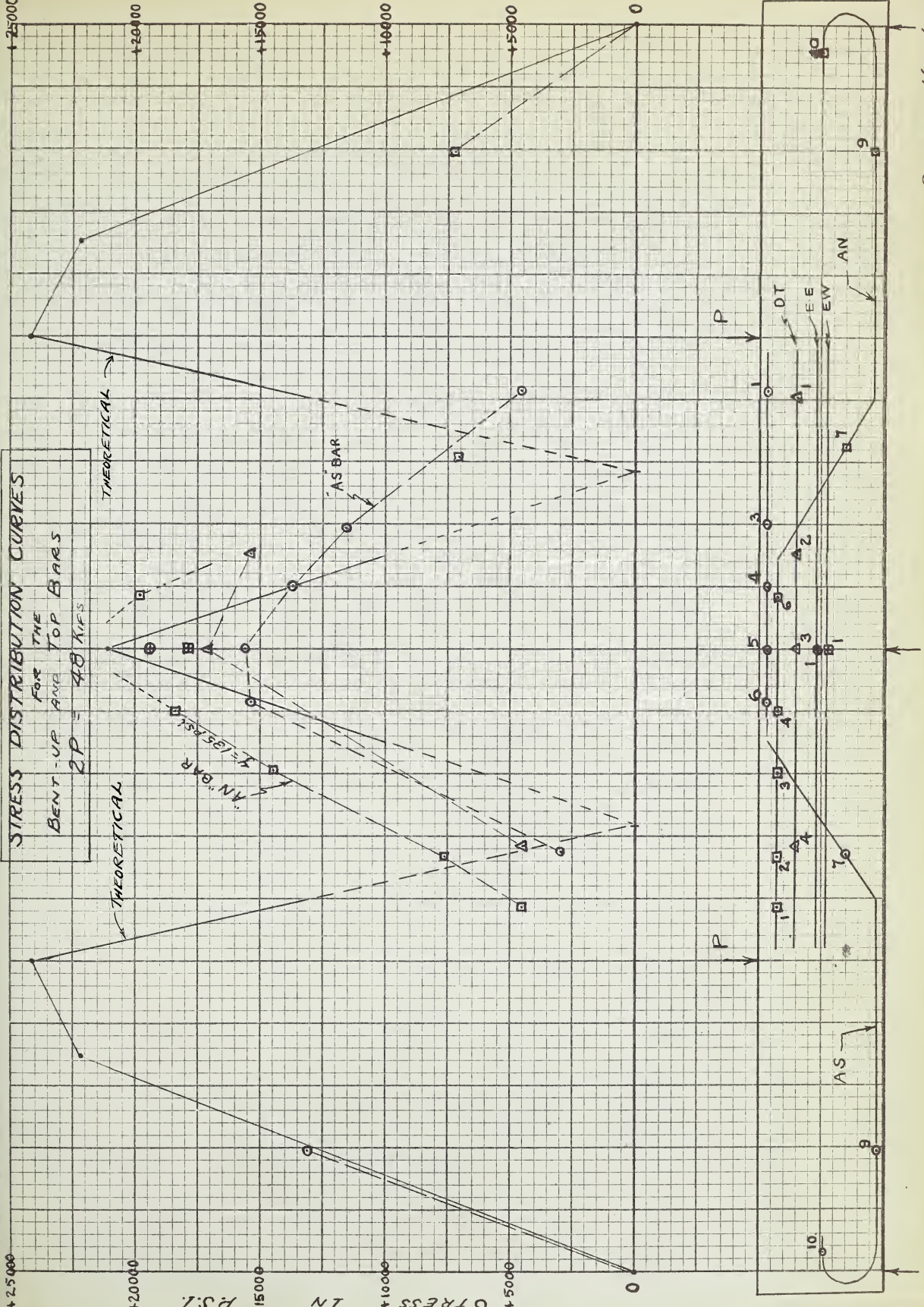




STRESS - DISTRIBUTION CURVES
FOR THE
BOTTOM REINFORCING BARS
WITH
LOAD $2P = 68$ KIPS

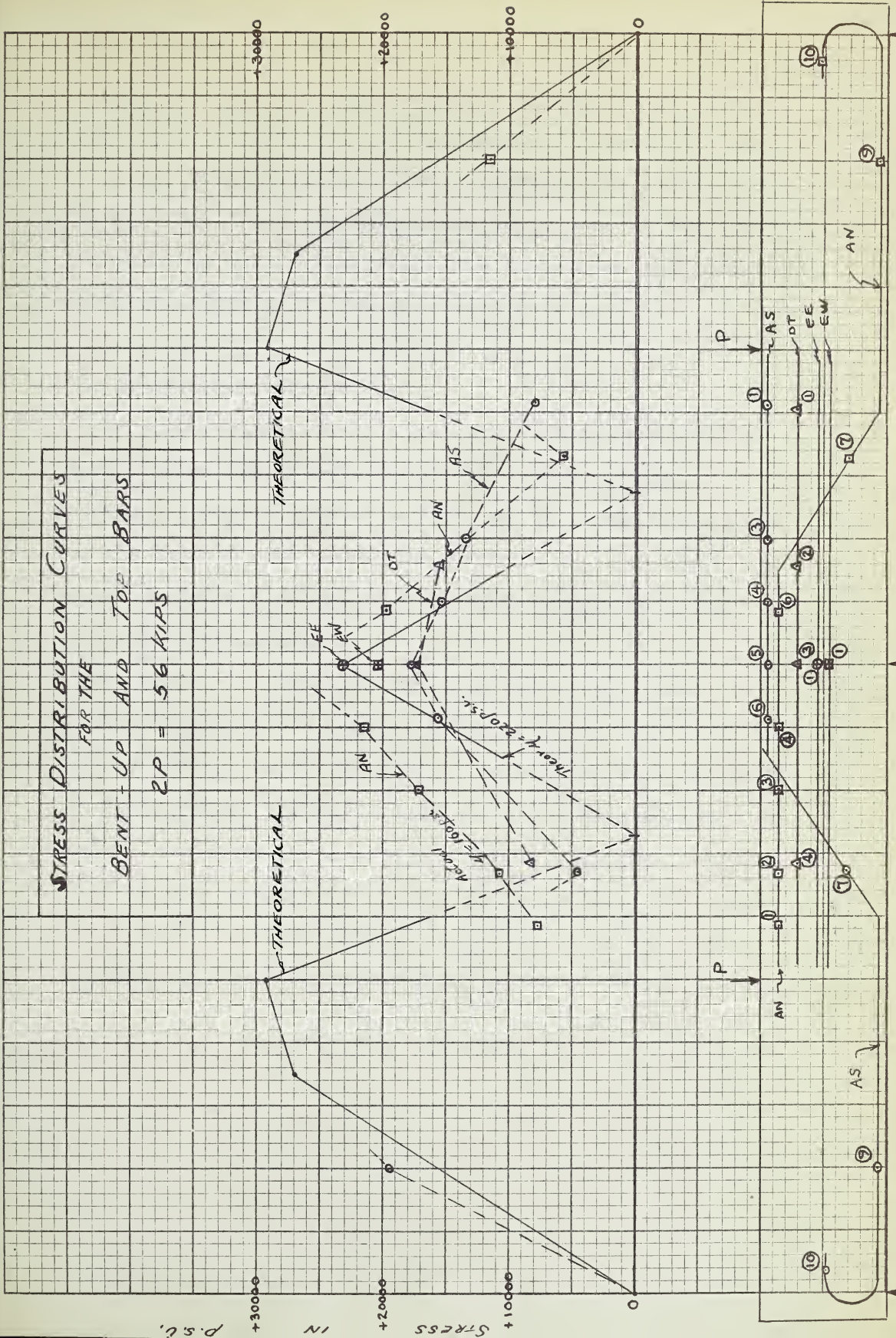


STRESS DISTRIBUTION CURVES
FOR THE
BENT-UP AND TOP BARS
 $2P = 48 \text{ KIPS}$



CONC. No. 10

STRESS DISTRIBUTION CURVES
FOR THE
BENT-UP AND TOP BARS
 $2P = 56 \text{ KIPS}$

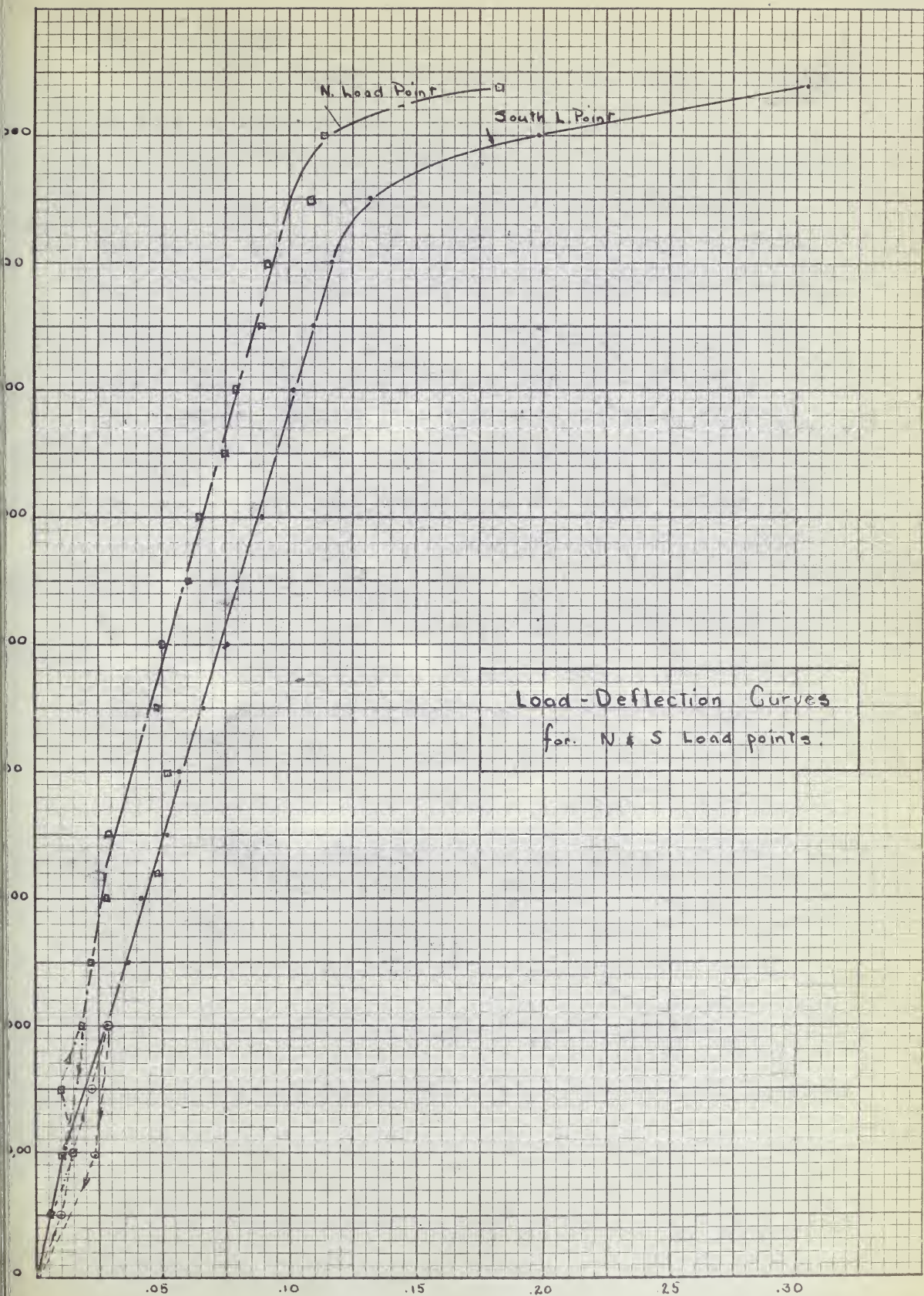


indicate that the ends of the hooks were not called upon for anchorage until very high stresses existed in the bars. one cannot obtain reliable figures on the value of hooks from the scant readings taken. However, the indication is that the hooks prevented failure in bond of the bottom reinforcing bars. The result was that these bars carried stresses right up to the yield point.

4. Load-Deflection Curves

These curves are a plot of the deflection of the beam at the load point against the various loads applied. The deflection plotted is the downward movement of the beam at the load-point relative to a straight line joining the top of the proving ring to the top of the sinking end-support.

The curves show the interesting feature of a "levelling-off" at a certain total applied load. This occurs for the South load point when $2P$ is equal to about 68 kips, and at the North load point for $2P$ about 72 kips. Before that time the curves are nearly straight. The curves thus clearly reflect the stress in the steel. At a load of 68 kips, the steel stress under the south load point was somewhat past the yield point value of 40,000 p.s.i.



DEFLECTION - ins.

GRAPH No. 12.

(g) Discussion

Taken as a whole, the beam test had certain weaknesses, as well as considerable value.

The number of strain gages used was barely sufficient to give a good indication of bond stresses throughout the beam. The actual effective number of gage locations was 20 to begin with because of their symmetrical placing, and this was reduced to ~~about~~ 18 by the spoiling of some gages. Of these a few could probably have been more strategically placed. For instance, the value of hooks could have been more accurately determined by placing a gage on the bar just before the hook itself. Also, it is regretted that not enough gages were available to allow placing some on the stirrups.

A more complete picture of the behaviour of the stresses was gained using the method described than was possible using other methods. Several events in the history of the load test which would otherwise be in doubt could be interpreted with the aid of the strain gage readings. Thus hair-cracks, although not visible, must have developed under the load-point when the total load was about 24 kips. At the design load of 32 kips the steel stresses were lower than the theoretical, and by a considerable amount at certain points. Anchorage of the top reinforcing bars first came into effect at a total load of 44 kips. Furthermore, this anchorage, by a straight

embedded length of 10 inches of bar in compression concrete, was insufficient to develop more than a maximum tensile stress of 24,000 p.s.i. in the bars, far below their capacity. If these bars had been hooked the capacity of the beam would undoubtedly have been increased. After the slippage of the top bars occurred, more moment was transferred to the quarter-points. The bottom bars, being anchored by hooks, were able to carry this load until their yield point was reached. The beam may be considered to have failed at this point, the load being about 68^k. The beam began to deflect rapidly at the load point and the tension crack which existed there increased in width. The fact that more load was taken (up to 75^k) was due to the greatly increased rate of loading necessary towards the end of the test.

The slip of the top bars also allowed a rapid widening of the tension crack which existed in the top concrete almost directly above the proving ring. This crack travelled downwards until it nearly traversed the beam. This probably explains the setting up of tension stresses in the compressive reinforcing over the centre-support which began at a total load of 56 kips. In fact, it is difficult to conceive any other reason for this, (since considerable negative moment existed over the centre support right up to the end of the test.)^{*k.k.}

The correlation of anchorage bond stresses measured in the beam with those found in the pull-out tests is

uncertain . This was because too few gauges were placed on the anchored portions of the bars in the beam. The few results computed indicate an agreement in the order of magnitude of bond stresses developed, but are not sufficient to differentiate the bond stresses due to tension and compression concrete.

The measured bond stresses due to shearing forces in the beam are lower than the theoretical as a rule. This was especially true of the top bars which maintained a rate of change of stress much lower than the theoretical. The agreement is better for the bottom tensile bars at high loads.

It is difficult to judge the requirements of the C.E.S.A. Code regarding bond in the light of the results of this test. It would appear that the factor of safety allowed in anchorage bond is not too high. Whether the same applies to the bond stresses allowed on hooks (125% of the value for straight embedment) cannot be said, as more data is required on the effectiveness of hooks. The allowable bond stress due to shearing forces would be too high for the strength of concrete used, if judged by this test. The computed theoretical bond stress on the top bars at the 56 kip load was only 220 p.s.i. (see graph 11) and yet, the bars were rapidly losing stress at this time. The allowable stress, according to the code, is 240 p.s.i. for 3000 pound concrete. A slip of the bars at this stress is surprising.

Possibly the fact that the top bars were spaced somewhat more closely in the poured beam than the Code allows had a detrimental effect on the development of bond strength.

Some of the above results are in agreement with results of simple beam tests carried out by the Bureau of Reclamation in Denver, U.S.A. They were described in an article * in the June, 1948 Journal of the American Concrete Institute. The tests showed the measured steel stresses in simple beams reinforced with one bar to be considerably below the theoretical values. The maximum bond stress due to shearing forces was found to be 580 p.s.i. for a $\frac{1}{4}$ " ϕ regular deformed bar.

*

"Laboratory Measurements of Stress Distribution in Reinforcing Steel," by McHenry and Walker.

VII.

CONCLUSIONS

This study of bond stresses, and the methods employed in its execution, though too limited to give proven results, has revealed some useful information. A summary of the most important indications follows.

(a) Pull-Out Tests

1. In pull-out tests of cylinders, the highest intensity of anchorage bond stress is developed at the loaded end of the bar. At low loads, the bond stress soon decreases to a negligible amount at some distance from the unloaded end of the bar. At high loads, especially after slippage begins, the bond stress tends to become more uniform.
2. Tension concrete in a pull-out test develops a higher intensity of anchorage bond stress than does compression concrete at the same load. The total bond developed is about the same in each case.

(b) Beam Test

1. The concrete in the reinforced concrete beam carried considerably more of the load than was assigned to it by the design formulae.
2. Hooks were very effective in their function as

anchorage to permit development of tensile stresses in bars.

3. Straight embedment of bars in compression concrete did not prove effective in developing tensile stresses in the bars.
4. Bond stresses due to shearing forces overcame the bond strength along the top bars at a very low value of bond stress. Close spacing of the top bars is suspected as the fault.

(c) Strain Measuring Technique

1. It was quite definitely shown that the employed method of measuring stresses inside concrete is reasonably satisfactory for laboratory work.
2. The method has some possibilities for the field testing of reinforced concrete structures under ideal conditions.
3. It was shown that a detailed picture of the steel stress distribution in a reinforced concrete structure under a wide range of loads can be gained by using the technique of electric strain gage measurement.

APPENDIXES

APPENDIX AWATERPROOFING OF ELECTRIC STRAIN GAGES(a) General Description

An electric strain gage consists of a small grid of very fine wire solidly held by strong glue between two pieces of thin paper. Its principle of operation in measuring strain depends on the fact that the electrical resistance of the grid of wire varies directly as the strain that the grid may undergo. If the gage is cemented firmly to a material, the grid of wires will suffer the same strains as the material does; by measuring the resistance changes of the grid by a sensitive galvanometer the actual strains in the material may be found. The strains are then converted to stresses by multiplying by the Modulus of Elasticity of the material.

The gages used in this work were SR 4 gages made by the Baldwin Southwark Locomotive Works, Philadelphia, U.S.A. The types available in the laboratory were A-1's and A-3's, with a gauge length of $\frac{15}{16}$ inch. A complete description of the method of applying these gages to steel surfaces is given by the makers in their Bulletin No. 179, "The SR 4 Bonded Resistance Wire Strain Gage", and need not be repeated here.

When properly attached, the strain gage grid is rigidly bonded to the steel (or other) surface, and yet is electrically insulated from it. It will work well as long

as it remains completely dry. If, however, the gage becomes damp or wet, then, since the paper and glue surrounding the grid wires are not moisture resistant, the electrical circuit through the grid is shorted or shunted through the bar or concrete and readings of resistance change are not reliable. The lower limit of the electrical resistance against this type of leakage is 50 meg. ohms between grid and bar for absolutely reliable results. Thus gages attached to steel bars immersed in water or wet concrete must be almost perfectly protected against entrance of moisture into the grid and against electrical leakage out of the grid, usually over long periods of time.

Several methods of doing this have been developed by others (2, 3, 4). In a paper to the A.C.I. (2), two investigators of the U.S. Bureau of Reclamation, Denver, have listed the requirements for an ideal type of SR 4 gage protection, and then have given a detailed description of two methods which they have successfully used. In order to carry out the bond tests, one of the above methods was tried and tested in the laboratory. The account which follows describes this method of the application and waterproofing of SR 4 gages on reinforcing bars, and its value and weaknesses.

(b) Application of the Gages to Reinforcing Bars.

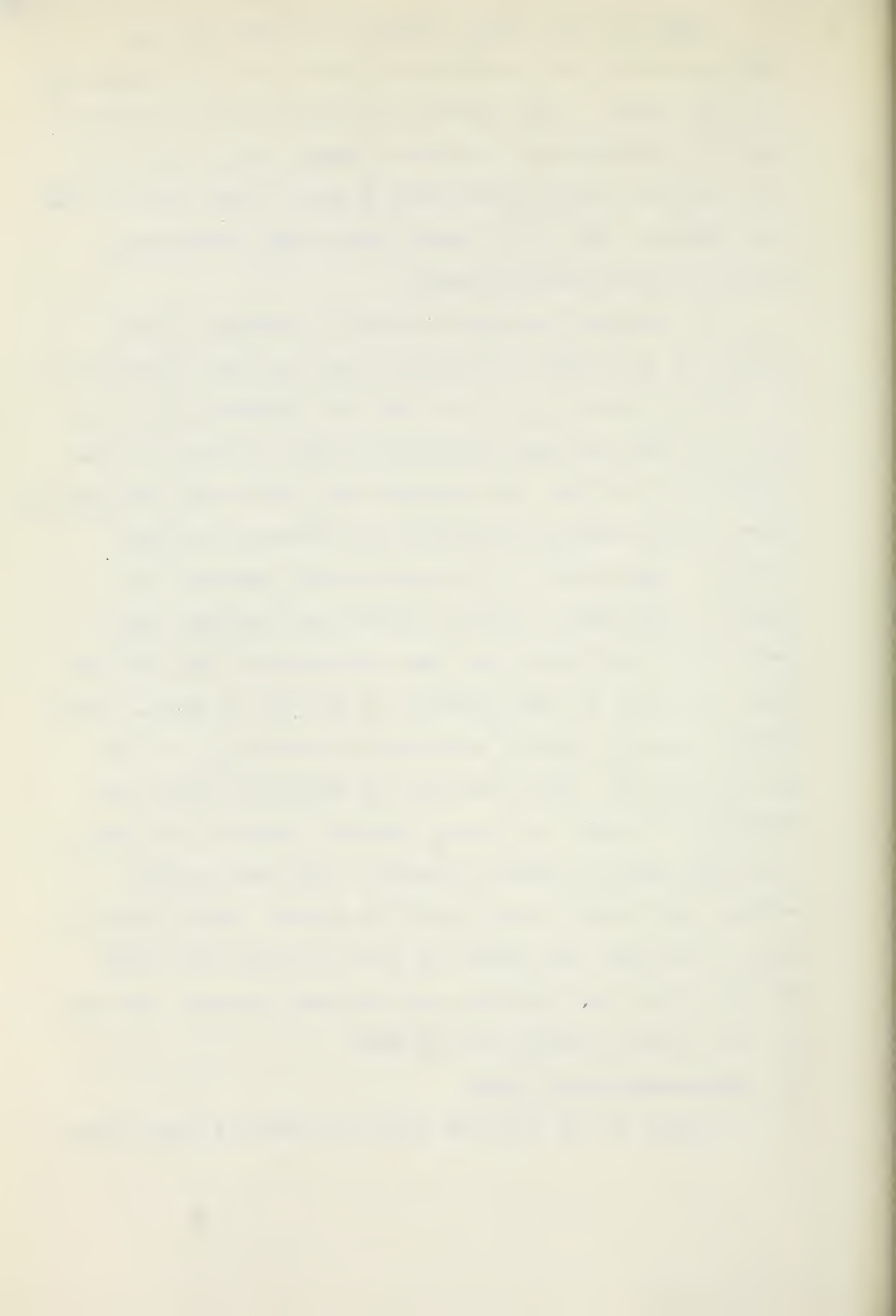
The method of attaching gages to reinforcing bars differs but slightly from the methods used for other steel shapes.

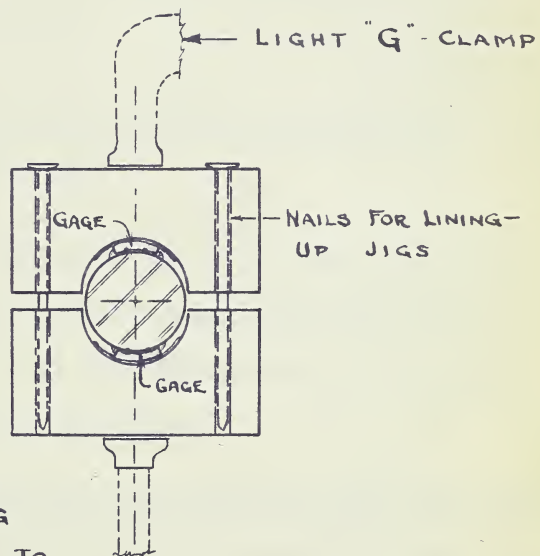
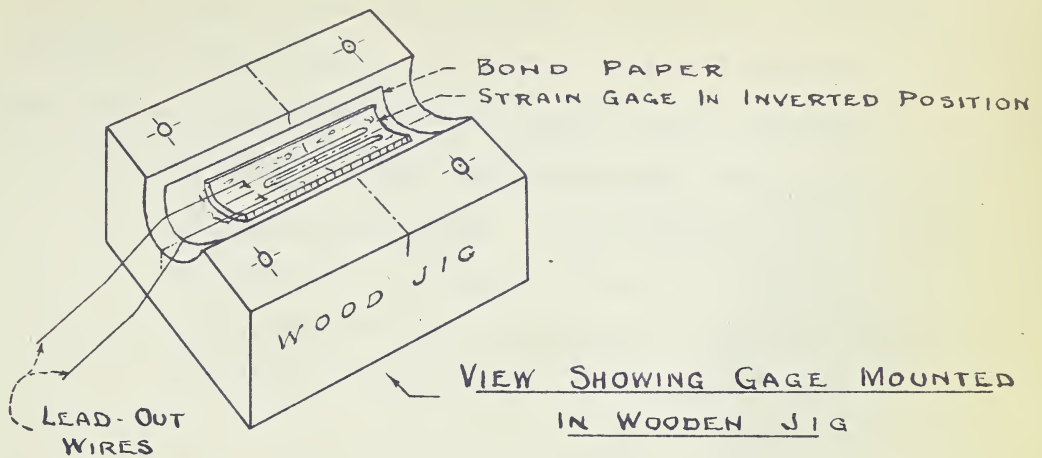
First of all, excess paper is cut from the gage right up to the felt protection, and to the glue holding the lead out wires. The surface at the gage location must be carefully cleaned and polished as usual, except that the area cleaned should extend about $\frac{1}{2}$ inch on all sides of the gage itself. This is to insure close bond between the steel and waterproofing agent.

In applying the gages to bars of circular cross-section it was found difficult to make the gage conform to the curved surface of the bar with the fingers and at the same time keep the gage centre-line truly parallel to the direction of stress. To overcome this difficulty the gage was given a predetermined curvature and direction by first gluing it very lightly in an upside-down position to a piece of bond paper, which was then glued solidly into a small wooden jig having the same curvature as the bar surface. The gage is then cemented to the bar as usual, being held in place by lightly clamping the wooden jig to the bar (see Fig. 8) This procedure is especially useful in fastening two gages on exactly opposite sides of the bar, with both gages to measure strains at the same cross-section, and truly in line with the stress. After about 16 hours of drying, the wooden jig can be pulled off, since the very light bond between paper and felt breaks, leaving the gage firmly attached to the bar.

(c) Waterproofing of Gages

As soon as the gage has dried at least 24 hours after





SECTIONAL VIEW SHOWING
TWO GAGES ATTACHED TO
OPPOSITE SIDES OF 5/8 BAR

SCALE: FULL SIZE

FIGURE 8

~~after~~ cementing (in moist atmospheres this process should be aided by heating the gages to about 170° F for an hour), the waterproofing process may begin. The following list of steps contains the minimum procedure necessary to obtain reasonably good results; failure to omit any one of them may be sufficient cause to spoil a gage location.

- (1) Test the leakage resistance and true resistance of the applied gage. Use a maximum test voltage of 45 volts to measure leakage resistance, and a wheatstone bridge to measure true resistance. A leakage resistance of less than 50 meg-ohms, and a true resistance varying by more than 1.0 ohm from the listed resistance indicates that the gage is of doubtful usefulness and should be replaced.
- (2) Clean all excess Dugco cement from the bar surrounding the gage.
- (3) Bend lead-out wires of the gage back over felt protection and fasten by cementing over them another small piece of felt about 1/3 the size of the original felt cover.
- (4) Solder the insulated lead wires to the gage leads. The insulation of these wires should be cut back by no more than $\frac{1}{2}$ inch for this size of gage. A good job of soldering, with no sharp projections,

must be done.

The insulated lead wires may be either the single or multi-strand type, preferably the former. The insulation itself must be a high quality rubber or plastic insulation, and should have a protect-sheath around it, preferably of rubber tubing. Lacquered fibre insulation will not do.

- (5) Apply first coat of 3-M Adhesive*, lifting the wires clear of the felt. This coat is important and should thoroughly cover the whole gage and about $\frac{1}{4}$ inch of the surrounding bar. The coat should be allowed to dry thoroughly (takes about 12 hours) before proceeding.
- (6) Apply a piece of electricians rubber tape about $\frac{3}{8}$ " square to the bar at the back end of the gage (opposite end to lead-outs), leaving up to $\frac{1}{8}$ " clearance between it and the felt. Then lower the insulated leads to this and paint on a second liberal coat of 3-M Adhesive. A thicker consistency of adhesive is better.
- (7) If the cover over the bare gage lead-outs and over the soldering is thin (most likely), cover these points with small pieces of electricians rubber tape.
- (8) Place another piece of $\frac{3}{8}$ " square electricians

* AIR-SETTING RUBBER CEMENT : 3M ADHESIVE No. EC-711,
AVAILABLE FROM " CANADIAN DUREX ABRASIVES LTD.,
CANAL ROAD, BRANTFORD, ONTARIO

tape over the wires at the back end of the gage,
plus a piece of friction tape. Then fasten the insulated
lead-out wires firmly to the bar by binding with one
 turn of No. 20 annealed steel wire about $\frac{1}{4}$ " from the end
 of the gage.

- (9) Apply the third and preferably fourth coats of
J-A Adhesive, using thicker consistencies, with
 drying between each coat. The point where the
 insulated wires leave the gage is the critical
 one, and the adhesive should be thoroughly worked-
 in there.

- (10) Apply a first heavy coat of Tygonite M-15-B Cement,
 using a small spatula. Thorough drying takes about
 12 hours over a radiator. *

- (11) Apply a second fairly heavy coat of Tygonite
Cement smoothing off the finished surface to a
stream-lined shape.

If the area of bond destroyed is not serious, then the
 the waterproofed gage may finally be wrapped with electricians
 tape to give further protection against the concrete. Also,
 if much time is available, the gages may be tested by
 immersing in water for 24 hours.

The above steps are for the water-proofing method
 called the "Tygonite Method" by the Bureau of Reclamation
 investigators, and first described by them. These authors
 performed steps 4, 6, 7, and 8 above before painting on

* HEAVY AIR-SETTING CEMENT,
 MADE BY U.S. STONEWARE,
 AKRON, OHIO.

any adhesive at all, i.e. they performed the electrical insulation before waterproofing and not at the same time. This required more thorough working-in of the adhesive, and was not successfully used by the author.

(d) Discussion of Method

The method as described is rather tedious when only a few gages are to be applied. About one week should be allowed for the application of the gages. However, it takes very little longer to waterproof twenty gages than it does to do one, and the whole process could be speeded up by applying heat up to 180° F for drying out the various coats.

This method is best suited to laboratory investigations, but could be applied to field tests if the concrete placement around the gages were under strict supervision. In pouring the test beam, the gages were not "pampered" when it came to placing and rodding the concrete.

One weakness of this method is that the electrical leakage resistance breaks down in time, depending on the amount of and thoroughness with which the various protective coatings are put on. In the laboratory, tests on pull-out cylinders completely immersed in water showed gages "holding-out" for one week, two weeks, four weeks, and then gradually beginning to leak. In the beam test, where the beam was kept only damp, better results were obtained. This was partially due to an improvement in

technique, but also due to less water being available in the concrete. It seems likely that in field tests, especially on concrete above ground level, the gages would read accurately up to a period of two months at least.

As mentioned before, other methods of waterproofing have been developed. A second method described by the Bureau of Reclamation involves vulcanizing a neoprene rubber housing over the gage to the steel bar. This method requires a special mold and furnace, and has apparently been used only on square bars, but it is more rugged, and holds promise for field tests according to the Bureau.

Another method is described in a paper in the S.E.S.A. Proceedings (3). The procedure is somewhat similar to the Tygonite Method, but Petrosene B Wax is used as the chief waterproofing agent. Long-term leakage resistance is claimed in a field test.

The results of all long-term tests are at present somewhat doubtful because of the tendency of the Duco cement holding the gage to the steel to creep. This of course causes an extra strain in the gage not undergone by the material, causing resistance-change readings to be not truly representative of strains in the test material. The above mentioned article describes procedures to attempt to eliminate such errors.

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APPENDIX B

DATA FOR PULL-OUT TESTS AND BEAM TEST

COMPUTED DATA FOR BEAM TEST

DATA: PULL-OUT TEST NO. 3

LOAD P	ELECTRIC STRAIN GAGE READINGS - MICRO-INS./INCH										
		1(a)	1(b)	2(a)	3(a)	3(b)	4(b)	5(b)			
lbs.											
110		5-568	1-620	5-892	6-336	4-040	5-480	1-183			
50		5- <u>570</u>	1- <u>630</u>	5- <u>892</u>	6- <u>338</u>	4- <u>040</u>	5-492	1-190	UNDERLINED NUMBERS TAKEN AS INITIAL READINGS		
1000		5-590	1-640	5-902	6-348	4-049	5-498	1-210			
1500		603	660	909	350	050	5-501? 511	1-200			
2000		628	680	911	350	052	5- <u>502</u>	1-200			
2500		650	708	918	358	060	508	202			
3000		681	739	928	364	061	520	—			
3500		710	770	930	365	067	518	1- <u>210</u>			
4000		762	810	940	371	070	530	—			
4500		800	850	953	378	078	522	210			
5000		846	890	972	380	080	529	—			
5500		882	938	989	383	085	530	—			
6000		930	983	1010	391	090	530	—			
6500		973	1031	1032	400	100	538	—			
7000		1029	1080	1064	410	110	551	218			
7500		1071	1130	1097	422	123	551	220			
8000		1126	1180	1140	440	140	554	220			
8500		1173	1230	1180	461	166	560	220			
9020		1238	1282	1230	491	192	583	222			
9500		1282	1332	1274	528	231	587	227			
10000		1338	1390	1332	573	279	595	230			
10520		1381	1432	1376	612	322	630	235			
11030		1439	1490	1430	660	370	641	240			
11500		1490	1541	1480	700	412	685	246			
11800		1530	1580	1574	726	442	673?	252			
12500		1618	1660	1605	800	520	721	274			
12750		—	—	—	—	—	—	—	ULTIMATE LOAD Y.P. OF BAR REACHED AT THIS LOAD. 1/8" SLIPPAGE AT 8000* LOAD.		
8000----- 7500		1243	1320	1372	630	387	682	220			
NOTE: LEAKAGE RESISTANCE OF GAGE 1(b) = 20 meg. ohms THIS GAGE SEEMS O.K.											
LEAKAGE RESISTANCE OF GAGE 5(b) = 200,000 ohms. GAGE OF DOUBTFUL VALUE.											

Feb 18, 1949.

BEAM TEST DATA

LOAD 2P					ELECTRIC STRAIN GAGE READINGS - MICRO-INS./IN.							
	TIME	TEMP.	PROVING RING READING		A S 1	A S 3	A S 4	A S 5	A S 6	A S 7	A S 9	A S 10
<i>lbs.</i>	<i>FRIDAY, MAR 25/49</i>	<i>°F.</i>	<i>$\frac{1}{10000}$ ins.</i>									
0	10:30 AM	72	70.5		4-802	5-548	4-489	4-570	5-580	4-468	5-516	2-890
0	2:30 PM	74	71.5		820	565	4-497	585	595	471	530	893
4000	3:05 -	74	80.5		818 825	572 581	510 510	603 607	611 618	482 487	537 541	903 963
8000	4:00 -	"	89.9		817 815	582	531	628	630	490	545	898
12000	4:30 -	75	99.2		809	573	528	638	640	490	557	900
16000	4:55	"	108.2		800 820	580 600	548 560	688	660	500	568	910
7920	5:25	"	90.3		839 828	600 584	550 540	691	658	510	553	910
0	5:50	76	71.0		840	593	538	645	635	520	610	935
8500	7:20	77	91.0		844	650	580	688	672	542	740	1022
16000	7:45	"	107.9		810	598	560	711 770	675	511	590	930
20000	8:15	"	116.7		859 805	646	607	793	710	578	580	912
24000	9:00	77	125.0		790	600	612	809	798	520	592	910
28000	9:45 PM	77	133.3		780	609	663	850	858	530	591	905
25500	SAT, MAR 26 8:45 AM	73	127.2		790	620	685	842	864	551	600	918
32000	9:35 -	74	141.3		770 780	670 673	732 740	896 904	960 970	550	636	920
36000	10:20 -	"	149.9		760 751	740 748	790 798	938	1021	549	637	910
40000	11:05 -	"	158.3		753	803 813	838 842	974 981	1048	549	669	918
44000	11:35 -	"	167.0		860 876	870 878	890	1038	1080	557	789	910
48000	12:40 PM	75	175.5		972 985	950	955	1103 1112	1108	570	968	918
52000	1:15 PM	"	183.7		1055 1070	1013	1010	1162	1148 1145	595	1066	914
56000	1:45 -	76	191.2		1085	1012	1006	1175 1159	1118	623	1160	920
60000	2:10 -	76	196.4		1070	950	937	1128	995	670	5-1300 6-329	916
64000	2:38 -	76	202.0		1058 1088	911	891	1098 1110	917	903 942	483	928
68000	3:05 -	"	204.8		1088	982	889	1104	883	1088	660	931
72000	3:25 -	77	210.8		1093	120 Failed	868	1090	838	1080	690	936
75000	3:45 -	"	216.2		Failed	—	—	—	807	1100	713	935
0	—	77	68.7									

NOTES:

28,000* load left overnight - 2500* lost overnight.

First visible fine cracks noticed at $\frac{1}{4}$ pts. and over the proving ring at load of 32000*.Additional fine tension cracks at $\frac{3}{8}$ pts., i.e., between end supports and load noticed at 56,000*. Continuous cranking necessary at 72000*.

BEAM TEST DATA

STRAIN GAGE READINGS - MICRO-INS./IN.													
10 2	A-N 1	A-N 2	A-N 3	A-N 4	A-N 6	A-N 7	A-N 9	A-N 10	B-S 4	B-S 5	B-S 6	B-N 1	B-N 2
Lbs.													
0	² -888	⁰ -210	³ -842	² -180	² -332	¹ -117	³ -302	¹ -145	⁰ -196	⁰ -669	² -832	⁰ -640	³ -480
0	² -898	⁰ -220	³ -850	² -189	330	117	307	¹ -142	178	⁶ -661	836	657	³ -490
100	² -891	² -214	854	200	340	120	² -343?	² -200?	162	⁶ -666 ⁶ -670	841	660	478
200	888	212	862	211	352	127	³ -321	¹ -148	169	690	920?	670	488
300	902	220	880	239	380	140	329	149	142	688	858	667	455
400	889	221	890	250	422	150	338	153	130	700	868	692	440
500	900	220	871	238	402	142	330	157	155	711	890	678	490
600	922	241	882	225	382	140	336	189	185	672	850	690	500
700	940	267	917	270	435	180	333	162	203	778	902	680	480
800	904	232	900	268	448	160	350	166	174	738	891	700	469
900	900	229	908	288	⁴ -60 ⁵ -512	173	371	188	153	738	904	695	437
1000	876	218	902	374	531	170	360	159	115	747	895	693	408
1100	859	206	919	430	572	170	365	159	105	820	905	705	400
1200	875	218	968	437	576	178	373	159	112	828	908	727	404
1300	854	210	1060	536	621	186	386	160	100	938	922	722	390
1400	850	220	1135	610	693	200	390	158	100	1158	934	728	381
1500	860	298	1200	670	770	223	409	160	087	1263	1000	723	377
1600	924	368	1262	732	842	352?	432	170	083	⁰ -1380 ¹ -410	1138	738	370
1700	1050	474	³ -1333 ⁴ -371	802	990	251	546	210	082	517	² -1298 ³ -334	753	374
1800	1056	495	⁴ -391	822	³ -41 ² -52	270	568	168	078	592	³ -409	771	360
1900	1157	576	462	910	990	308	695	168	088	690	500	810	370
2000	1180	562	400	845	946	338	847	171	107	814	628	881	381
2100	1140	535	348	820	888	408	1028	170	207	946	743	1060	400
2200	1132	520	320	883	845	658	³ -1194 ⁴ -237	178	422	¹ -1212 ² -248	930	1161	430
2300	Failed	533	301	950	828	746	⁴ -300	177	550	Failed	950	1140	508
2400	—	—	288	—	825	745	310	175	631	—	988	1148	842

[illegible]

BEAM TEST - COMPUTED DATA

[illegible]

BEAM TEST—COMPUTED DATA

BEAM TEST - COMPUTED DATA

LOAD P	UNIT STRESS AT GAGE POINT - P. S. I.												
	A-N 4	A-S 5	A-S 6	A-N 6	A-S 7	A-N 7	A-S 9	A-N 9	A-S 10	A-N 10	B-N 1	B-N 2	B-N 3
KIPS													
0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	+330	+540	+480	+300	+330	+90	+210	?	0	?	+90	-360	-240
4	+660	+1290	+1050	+660	+570	+300	+450	+42	0	0	+390	-60	-390
6	+1500	+1590	+1350	+1500	+570	+690	810	+660	0	0	+300	-1050	-740
8	1830	3090	1950	2760	870	990	1140	930	0	+90	450	-1500	-1740
396	1470	3180	1890	2160	1170	750	690	690	0	+210	630	0	-540
0	1080	1800	1200	1560	1470	690	?	870	+750?	+870?	990	+300	+360
4.25	2430	3090	2310	3150	2130	1890	?	780	?	+360	690	-300	-330
8	2370	3780	2400	3540	1200	1290	1800	1290	+600?	+480?	1290	-630	-1020
10	2970	6240	3450	5460	3210	1680	1500	1920	+60	+1140?	1140	-1590	-1980
12	5550	6720	6090	6030	1470	1590	1860	1590	0	+270	1080	-1860	-2970
14	7230	7950	7890	7260	1770	1590	1830	1740	-150	270	1440	-2700	-3480
12.75	7440	7710	8070	7380	2400	1800	2100	1980	+240	270	2100	-2580	-3480
16	10400	9330	10900	8730	2370	1830	3180	2370	+300	300	1950	-3000	-4080
18	12600	10600	12800	11900	2340	2070	3210	2490	0	240	2130	-3270	-4680
20	14400	11700	13600	13200	2340	2490	4170	3060	+240	300	1980	-3390	-4860
22	16300	13600	14600	15400	2580	3180	7770	3550	0	600	2430	-3600	-4350
24	18400	15600	15400	19800	2970	7050?	13100	7170	+240	?	2880	-3480	-3870
26	19000	17300	16600	18300	3720	4020	16100	7830	+120	540	3420	-3900	-3720
28	21600	17700	15700	19800	4560	5730	19500	11600	+300	540	4590	-3600	-2580
30	19700	16300	12000	18500	5970	6630	23100	16200	+180	630	6720	-3720	+720
32	18900	15400	9460	16700	13000	8730	27700	21600	+540	600	12100	-2700	+4020
34	20800	15600	8640	15400	18500	16200	33000	26600	+630	840	15100	-1800	+9270
36	22800	15100	7290	15000	18300	18900	33900	28500	+780	810	14500	+540	+15000
37.5	—	—	6360	14800	18900	18800	34600	28800	+750	750	14700	+10600	+23500

BEAM TEST - COMPUTED DATA

LOAD P	UNIT STRESS AT SECTION - P. S. 2.												
	B-S 4	B-N 4	B-S 5	B-N 5	B-S 6	B-N 6	C-S 1	C-S 2	C-N 2	C-S 3	C-N 3	C-S 4	C-N 4
KIPS													
0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	-480	+90	+210	+450	+150	+300	+300	+420	+360	+270	+240	+60	+30
4	-270	-360	+870	+900	?	+720	+180	+1260	+840	+360	+360	+120	-60
6	-1080	-600	+810	+1050	+660	780	-300	960	1560	+1110	960	+420	+150
8	-1440	-1350	1170	1200	960	930	-150	1320	2070	1200	1200	420	+120
3.96	-690	-120	1500	1200	1620	960	-30	720	1120	690	900	120	+270
0	+210	+150	330	300	420	960	+300	1140	1080	780	720	600	420
4.25	+750?	-240	?	1200	1980	1320	+600	990	1680	960	1200	1200	420
8	-120	-750	2310	1860	1650	2100	+120	1560	1830	1440	1500	360	390
10	-750	-1290	2310	2100	2020	1590	+300	2160	2880	2010	2760	420	150
12	-1890	-1800	2580	2400	1770	1560	± 0	2760	3180	2070	2400	210	330
14	-2190	-1980	4770	3900	2070	2130	+180	4590	4380	2760	3000	210	390
12.75	-1980	-1800	5010	3900	2160	2220	420	5760	5250	2940	2940	210	360
16	-2340	-2250	8310	9000	2580	2490	900	9960	11400	4320	3870	660	540
18	-2340	-2610	14900	12600	2940	2760	750	14600	15500	6570	4050	270	510
20	-2730	-2850	18100	15300	4920	3000	960	17500	19400	8850	5760	450	630
22	-2850	-2850	21600	18100	9060	3660	990	20200	22700	11600	8290	660	600
24	-2880	-3000	24800	20600	13900	6000	1590	23100	26300	14600	10600	1410	1290
26	-3000	-3150	27000	22600	16100	8010	2130	25500	28600	16800	13100	1290	1350
28	-2700	-2970	30000	25200	18800	11100	4350	28200	31900	19400	16100	1860	1410
30	-2130	-2550	33700	28700	22700	15100	9540	33100	35600	26600	19300	6360	1200
32	-770	-1720	37600	32300	26100	20100	15300	35700	39400	26900	23200	9570	1740
34	+7320	+3450	45600	36600	31700	24700	19700	42000	42300	35500	26900	10900	5790
36	+11200	+5910	—	38600	32300	25900	20300	—	44300	34700	28000	9720	7200
37.5	+13600	+16300	—	—	33500	30200	21400	—	—	41300	36700	10600	7530

BEAM TEST - COMPUTATION OF STRESSES

LOAD 2P	DIRECT STRESSES					BOND STRESSES								
	R	2P-R	$f_s - \frac{1}{4} \text{ pt.}$	$f_s' - \frac{1}{4} \text{ pt.}$	$f_s' = 0.715 \times (P-R)$	TOTAL STRESS @	TOTAL STRESS @	TOTAL STRESS @	DIFF. IN STRESS	DIFF. IN STRESS	DIFF. IN STRESS	AVG. BOND STRESS	AVG. BOND STRESS	AVG. BOND STRESS
KIPS	lbs.	lbs.	$f_s = 147(2P)$ p.s.i.	$f_s' = 0.715 \times (P-R)$ p.s.i.	A-N-1	A-N-2	A-N-3	O-A-N-1	AN1-AN2	AN2-AN3	O-1	1-2	2-3	
					#	#	#	#	#	#	BOND AREA = 6.5 in. ²	BOND AREA = 20 in. ²	BOND AREA = 13.0 in. ²	
0	0	0	0	0							p.s.i.	p.s.i.	p.s.i.	
4	2880	1120	1870	-630							NOTE: BOND AREA DESTROYED BY GAGE TAKEN AS 3.0 in. ² PER GAGE.			
8	5900	2100	3510	-1360										
12	8880	3120	5210	-2060										
16	11750	4250	7100	-2680										
7.92	6030	1890	3160											
0	0	0	0	0										
850	6420	2180	3640											
16	11800	4200	7020	-2720										
20	14650	5350	8930	-3320										
24	17300	6700	11200	-3790										
28	20000	8000	13400	-4290										
25.5	18050	7450	12400	-3790										
32	22500	9500	15900	-4650	-410	-90	+1950	-410	-320	+2040	-60	-50	+160	
36	25300	10700	17900	-5220	-450	0	+2650	-450	-450	+2650	-70	-60	+200	
40	28000	12000	20000	-5720	-350	+720	+3260	-350	+1070	+2540	-50	+150	+200	
44	30700	13300	22200	-6220	+240	+1380	+3830	+240	+1140	+2450	+40	+160	+190	
48	33500	14500	24200	-6790	+1410	+2360	+4490	+1410	+950	+2130	+220	+140	+160	
52	36100	15900	26600	-7220	+1470	+2560	+4680	+1470	+1090	+2120	+230	+160	+160	
56	38500	17500	29200	-7510	+2410	+3310	+5330	+2410	+900	+2020	+370	+130	+160	
60	40200	19800	33000	-7290	+2620	+3180	+4780	+2620	+580	+1600	+400	+80	+120	
64	41900	22100	36900	-7080	+2250	+2930	+4280	+2250	+680	+1350	+350	+100	+100	
68	42800	25200	42100	-6290	+2180	+2790	+4030	+2180	+610	+1240	+340	+90	+100	
72	44700	27300	45600	-6220	—	+2810	+3840	—	—	+1030	—	—	+80	
75	46500	28500	47600	-6430	—	—	+3710	—	—	—	—	—	—	

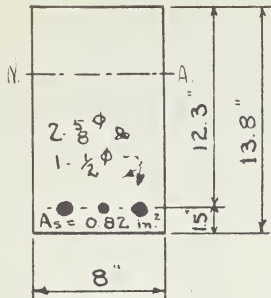
APPENDIX C

COMPUTATION OF THEORETICAL STRESSES

END OF APPENDIX

INVESTIGATION OF STRESSES AT LOAD POINT

NOTE: DIAGRAMS GIVE DETAILS OF BEAM AS ACTUALLY POURED



SECTION AT LOAD POINT

STRENGTH OF CONCRETE TAKEN AS 3000 p.s.i. FROM CYLINDER TESTS.
VALUE OF "n" ASSUMED AS 9
R.C. HANDBOOK TABLES USED

CASE I - CRACKED TO N.A.

$$m = q = \frac{n A_s}{b d} = \frac{9 \times 0.82}{8 \times 12.3} = 0.075$$

FROM TABLE II, $K = 0.321$

" " 13, $j = 0.89$

$$f_s = \frac{M}{j d A_s} \quad M_{\text{LOAD POINT}} = 1.25(2P-R) \text{ ' '}$$

$$f_s = \frac{1.25(2P-R) \times 12}{0.89 \times 12.3 \times 0.82} = 1.67(2P-R) \text{ p.s.i.}$$

CASE II - NO CRACKING OF SECTION

$$\text{SHIFT OF N.A.} = \frac{74 \times 5.4}{117.8} = 0.34" \text{ DOWN}$$

$$I_{\text{N.A.}} = 1940 \text{ in.}^4$$

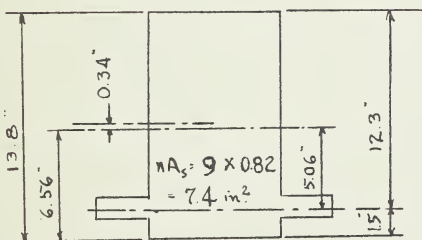
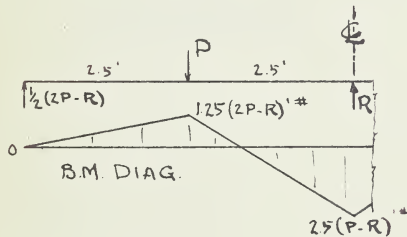
$$f_c = \frac{M y}{I}, \quad y = 5.06 \text{ in.}$$

$$= \frac{1.25(2P-R) \times 12 \times 5.06}{1940}$$

$$f_s = n f_c$$

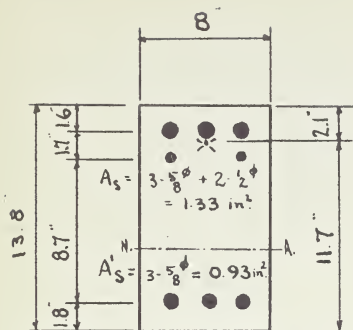
$$= \frac{9 \times 1.25(2P-R) \times 12 \times 5.06}{1940}$$

$$= 0.352(2P-R) \text{ p.s.i.}$$



TRANSFORMED SECTION
NO CRACKING

INVESTIGATION OF STEEL STRESSES AT Φ - CRACKING TO N.A -



SECTION AT Φ
OR AT
7" TO R. & L. OF Φ

$$C. of G. of Tens. Steel = \frac{4.0 \times 1.7}{1.33} = 0.51" \text{ below } \Phi \pm$$

$$m = \frac{nA_s}{bd} + \frac{(n-1)A'_s}{bd} = \frac{9 \times 1.33}{8 \times 11.7} + \frac{8 \times 0.93}{8 \times 11.7} = \frac{0.128}{0.080} = 0.208$$

$$q = \frac{nA_s}{bd} + \frac{(n-1)A'_s}{bd} = 0.128 + 0.080 \times \frac{1.8}{11.7} = \frac{0.128}{0.012} = 0.140$$

FROM TABLE 11, $k = 0.360$

$$\frac{j}{k} = \frac{(n-1)A'_s}{bd} = \frac{0.080}{0.360} = 0.22, \quad \frac{l}{k} \times \frac{d'}{d} = \frac{1}{0.360} \times \frac{1.8}{11.7} = 0.427$$

FROM TABLE 12, $z = 0.35$

" " 13, $j = 0.87$

$$f_s = \frac{M}{j d A_s}, \quad M_{\Phi} = 2.5(P-R)'M \quad (\text{See prev. sheet})$$

$$f_s = \frac{2.5(P-R) \times 12}{0.87 \times 11.7 \times 1.33} = \underline{\underline{2.22(P-R) \text{ p.s.i.}}}$$

$$\text{Also, } f'_s = f_s \times \frac{k - \frac{d'}{d}}{1 - k}$$

$$= f_s \times \frac{0.36 - 0.154}{0.64} = 0.322 f_s$$

$$f'_s = 2.22(P-R)0.322 = \underline{\underline{0.715(P-R) \text{ p.s.i.}}}$$

INVESTIGATION OF CONCRETE STRESSES AT Φ

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k} = \frac{2.22(P-R)}{9} \times \frac{0.36}{0.64} = \underline{\underline{0.139(P-R) \text{ p.s.i.}}}$$

$$\begin{aligned} \text{Max comp. stress (at 56" Load)} &= 0.139 \times 10,500 \\ &= \underline{\underline{1460 \text{ p.s.i.}}} \quad \text{MODERATE / STRESS} \end{aligned}$$

INVESTIGATION OF f'_s STRESSES AT 7" TO R. & L. OF Φ

$$M = 2.5P - 2.208R$$

$$= 2.5(P - 0.884R)$$

$$f'_s = \underline{\underline{0.715(P - 0.884R)}}$$

STRESS DISTRIBUTION ALONG BARS AT 48,000 # LOAD

1. FOR "B" - BARS :

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STEEL STRESS AT $\frac{1}{4}$ PT. - COMP SHEET ~~77~~

$$f_s = 1.67(2P-R) = 1.67 \times 14,500 = \underline{24,200 \text{ p.s.i.}}$$

STEEL STRESS AT $\frac{1}{8}$ PT.:

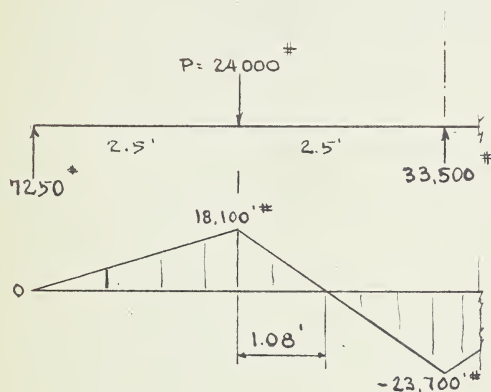
$$M = \frac{1.25}{2.50} \times 18,100 = 9050 \text{ #}$$

$$f_s = \frac{M}{j d A_s} = \frac{9050 \times 12}{0.89 \times 12.3 \times 0.62} = \underline{16,000 \text{ p.s.i.}}$$

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COMP STEEL STRESS @ $\frac{1}{2}$: SEE COMP. SHEET ~~77~~

$$f_s = 0.715(P-R) = 0.715(9500) = \underline{6,790 \text{ p.s.i.}}$$



B.M. DIAGRAM

DATA:

$$2P = 48,000 \text{ #}$$

$$R = 33,500 \text{ #}$$

$$2P-R = 14,500 \text{ #}$$

$$\frac{1}{2}(2P-R) = 7,250 \text{ #}$$

$$P-R = -9,500 \text{ #}$$

2. FOR "A" BARS :

STEEL STRESS AT $\frac{1}{4}$ PT. - AS FOR "B" BARS ABOVE.STEEL STRESS AT $\frac{1}{2}$:

$$f_s = 2.22(P-R) = 2.22(9500) = \underline{21,100 \text{ p.s.i.}}$$

STRESSES AT 9" TO R.B.L. OF $\frac{1}{2}$ - WHERE BARS BEND DOWN:

$$M = -23,700 \times \frac{0.67}{1.42} \text{ #}$$

$$f_s = 21,100 \times \frac{0.67}{1.42} = \underline{9,950 \text{ p.s.i.}}$$

CHECK ACTUAL BOND STRESS ON "A" BARS

- SEE GRAPH 10

DIFF. IN STRESS BETWEEN GAGES A-N-3 & A-N-4:

$$18,400 - 14,500 = 3,900 \text{ p.s.i.}$$

$$\text{TOTAL BOND DEVELOPED} = 3,900 \times 0.31 = 1,210 \text{ #}$$

$$\text{BOND AREA} = (6 \times 2.0) - 3 \text{ in}^2 (\text{FOR GAGE}) = 9 \text{ in}^2$$

$$\text{ACTUAL BOND STRESS DUE TO S.F.} = \frac{1,210}{9.0} = 135 \text{ p.s.i.}$$

COMPUTATION SHEET "C"

B29756